Development of a Standard Design Algorithm for Intze Tank

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by

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CERTIFICATE

It is certified that this work entitled **Development of a Standard Design Algorithm for Intze Tank** by *Ajeet kumar* has been carried out under our supervision and this work has not been submitted elsewhere for a degree.

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ABSTRACT

In the present work, an attempt is to made to develop a standard design algorithm for Intze tanks. The entire superstructure and substructure of the water tank is divided into three parts, viz. the tank, the supporting frame and the foundation. The effects of wind and earthquake are also considered for the design of water tank. The analysis of the tank is done into two stages - the membrane analysis and the continuity analysis. An approximate method is used for the analysis of staging type-supporting frame, whereas, a method based on thin plate theory is used for the analysis of annular type foundation. Finally, an algorithm is developed based on the review of available codal recommendations, analysis methodologies and design data so that a standard and concise design basis is available.

CONTENTS

LIST OF TABLES

LIST OF FIGURES

List of Notations

$A_{s1\,.\,s12}$	Area of reinforcement (membrane analysis)
A_{Φ}	Cross-section area of bar
A_{b1}	Area of cross-section of ring beam B_1
A_{sd}	Side face reinforcement
$A_{1c...7c}$	Area of reinforcement (continuity analysis)
A_c	Area of cross section of column
A_f	Area of foundation
A_p	Area of circumference of column
A_{ar}	Area of annular raft
b_1	Width of ring beam B_1
b_2	Width of ring beam B_2
b_3	Width of ring beam B_3
b_r	Width of brace
D	Diameter of container
D_0	Inner diameter of staiging
d_1	Depth of ring beam B_1
d_2	Depth of ring beam B_2
d_3	Depth of ring beam B_3
d_r	Depth of brace
d_c	Diameter of columns
d_i	Inner diameter of base raft
d_0	Outer diameter of base raft

d_s	Depth of raft slab
E_c	Modulus of elasticity of the columns
E_b	Modulus of elasticity of the braces
h	Height of container
h_0	Height of conical dome
h_1	Height of upper spherical dome
h_2	Height of bottom spherical dome
I_b	Moment of inertia of bracing beam
I_c	Moment of inertia of column
I_{rs}	Moment of inertia of slab about a diametrical axis
j	Lever arm factor .
M_{r0}, M_{r1}	Radial moment of raft slab
$M_{\theta 0}, M_{\theta 1}$	Circumferential moment of raft slab
m	Modular ratio
n	Number of columns
n_1	Number of panels
$N_{2....}$	Number of bars
P	Force pr sq m of top dome
p_y	Wind pressure
q	Bearing pressure of soil
R_1	Radius of top dome
R_2	Radius of bottom dome
R^1	Design factor

R_0	Mean radius of raft slab
r	Percentage steel distribution
r_i	Inner radius of raft slab
r_0	Outer radius of raft slab
$S_{1...5}$	Spacing of reinforcement (membrane analysis)
$S_{1c....7c}$	Spacing of reinforcement (continuity analysis)
T_1	Meridional thrust at the edge (top dome)
T_2	Maximum hoop force (top dome)
T_3	Meridional thrust (conical dome)
T_4	Hoop tension (conical dome)
T_5	Meridional thrust (bottom dome)
t	Thickenss of top dome
$t_{w\ ave}$	Average thickness of cylindrical wall
t_{con}	Thickness of conical dome
V	Volume of container
$W_{1\ ..28}$	Weight
x	Height of each panels
ϕ_0	Inclination of conical dome with horizontal
ϕ	Semi-central angle of top dome subtends at its centre of curvature
ϕ_1	Semi-central angle of bottom dome subtends at its centre of curvature
γ_c	Unit weight of concrete
σ_{md}	Meridional stress (top dome)
σ_{hd}	Maximum hoop stress (top dome)

σ_{st}	Permissible stress in high yield strength deformed bars
σ_{tc}	Allowable tensile stress of concrete
σ_{mcom}	Meridional stress (conical dome)
σ_{mbd}	Meridional stress (bottom dome)
σ_{hbd}	Hoop stress (bottom dome)
σ_{hbb}	Hoop stress (ring beam B_2)
σ_{bd1}	Bending stress (top dome)
β	Shape factor of tank.

Chapter 1.

Introduction

1.1 General

Out of the several shapes used in the water tanks, Intze tank has gained some popularity because of its dominant membrane action. Structurally and architecturally, it has some advantage while making the best possible use of circular shapes. The intersections between the elements when made symmetrical result in only minor perturbations. The main advantage of intze tank is that the outward thrust from the top of the conical part is resisted by the middle ring beam while the resultant of the inward thrust from the bottom of conical dome and the outward thrust from the bottom dome are resisted by lower ring beam. The proportions of the conical dome and bottom dome are so arranged that the outward thrust from the bottom dome is balanced by the inward thrust due to the conical dome.

Outstanding progress has been achieved in the field of devising better design solutions, for intze tank in the last four decades at the advent of the electronic digital computer. A conscious engineer has to investigate several alternative designs, and, has to select the best one among them. Many possibilities obviously exist, with clearly defined extremes. One extreme is to utilize the computer capability to fullest and 'automate' this search; the other extreme is to utilize human intuition in an 'interactive manner' to guide the computer in its calculations. A definite need exists for development of a standard

design algorithm based on the current codal recommendation in order to help the professionals.

1.2 Literature Review

Some published literatures are available for the accurate analysis and design of Intze tanks. *Rao* [1] has reviewed the effect of container geometry on the wind, earthquake forces and moments on the staging and foundation of the tank to achieve minimum weight or material content of the container. *Arya* [2] made an attempt to simplify the problem by setting down the many expressions involved in a convenient form, and, tabulating the numerical data for various shells in terms of dimensionless parameters suitable for use with any units and any combinations of shells or applies loads. *Sameer and Jain* [3] proposed a simple approximate procedure to estimate the lateral stiffness of the tank staging. These are based on the portal frame and moment distribution methods, which have been suitably modified to account for the bracing flexibilty and three-dimensional behavior of the structure. *Jain and Singh* [4] presented a short and efficient programme for the accurate analysis of Intze tank. The method involves membrane analysis followed by an effect of continuity analysis. *Arya* [5] proposed a ring beam on a solid or an annular raft with the object of minimizing the moments in the foundation raft. He also calculated radial and tangential moments in a solid or an annular raft under combined vertical and lateral loads. *Rao* [6] presented data in the form of graphs, which simplify the analysis of annular foundation for structures like water tanks. *Jain and Krishna* [7] proposed that the Intze type tank is the economical one in the capacity range

between 225 kl and 1800 kl. Some designs for various capacities of Intze t

available at *Uttar Pradesh Rajkiya Nirman Nigam Limited,* Lucknow[8].

1.3 Scope of the Present work

In this study, a standard design algorithm for the de

tank is developed based on the review of available codal recommendati

methodologies and design data.

Chapter 2

Preliminary Design

2.1 Introduction

In the present work, emphasis has been laid on the preliminary design of intze water tank structure, taking into account its strength and stability, for the given tank-capacity. The main purpose of preliminary design is to arrive at the different relevant dimensions of the different components of intze tank, in order to make a start in the design process. It also helps in obtaining the needed information for making preliminary review for the feasibility of constructing a tank in respect of both technical and financial aspects, at the preliminary stage. The process involves approximate and simple analysis for structure based on certain assumptions, current practice, codal recommendation and past experience.

The complete water tank structure can be considered as a combination of the three components viz., container or tank, staging or supporting frame and foundation (Fig. 1). Commonly adopted practice for the preliminary design is discussed in sequence in the following.

2.2 Preliminary Selection of Container-Dimensions

2.2.1. Using the empirical formula suggested by Rao [1]

Referring to the notation given in Fig 2. The capacity V of the container is given by

$$V = K_v D^3 \qquad (2.1)$$

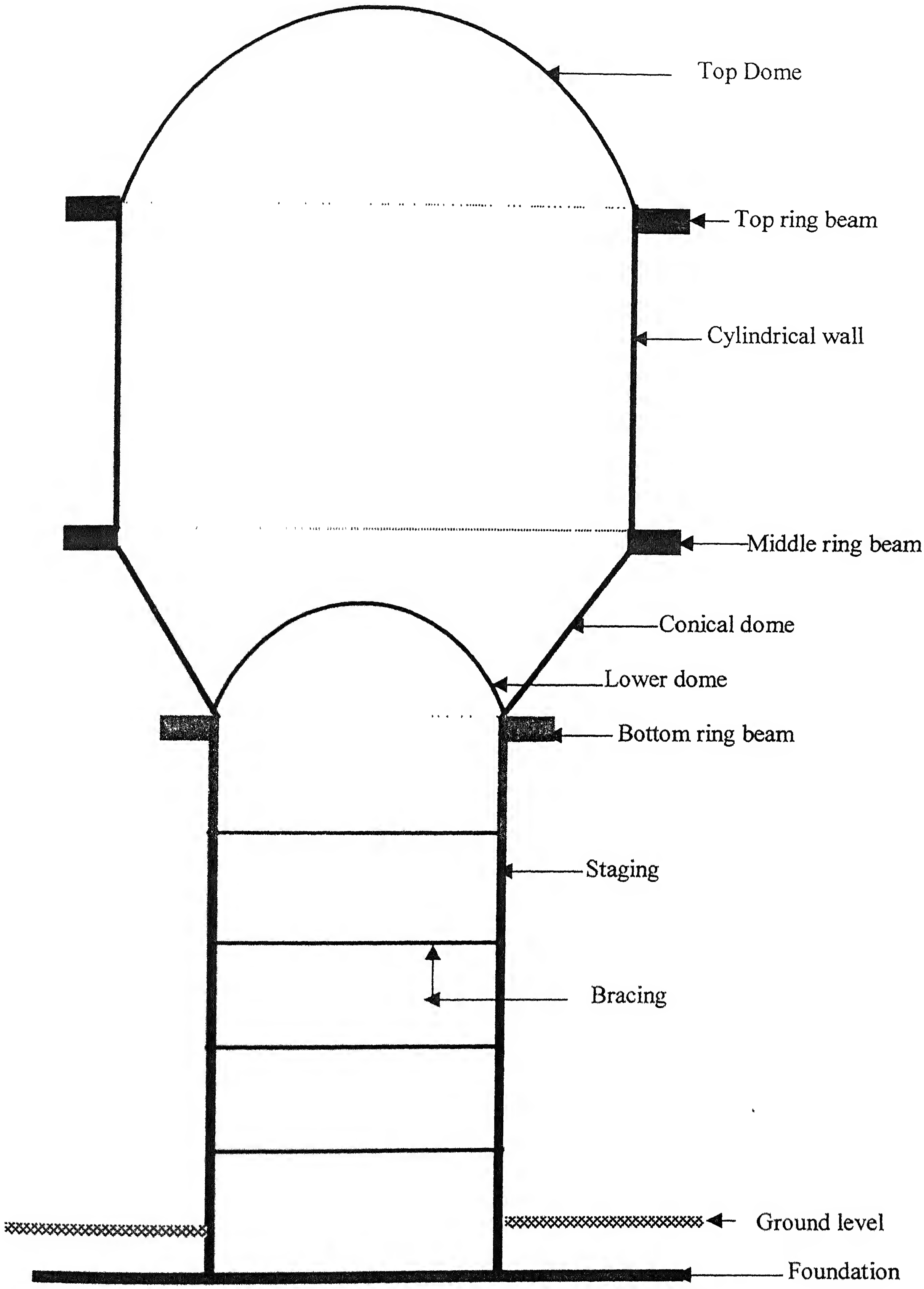


Fig. 1 Intze Tank

Where,

$$K_v = \frac{\pi}{4}\left[K_2 + \frac{K_5}{3}\left(1 + K_3 + K_3^2\right) - \frac{K_4}{6}(3K_3^2 + 4K_4^2)\right]$$

The parameters, K_1 to K_5 characterize the geometry of the tank as illustrated in Fig. 2. The parameter K_5 related to the slope of the cone, ϕ_0 and K_3 is given by

$$K_5 = 0.5(1 - K_3)\tan\phi_0$$

The magnitude of K_v depends on the parameter K_2 to K_5. Different authors suggested values of the parameters, which give economical container design. The typical values are presented in Table 1.

2.2.2 According to Jain & Jai Krishna [7]:

The diameter of the bottom dome should be about 70% of the diameter of the container. For economy, the inclination of conical dome may be 50^0 to 55^0 with the horizontal. The rise of the domes should be about 1/5 to 1/8 times the diameter of the tank.

Based on the consideration of the above approximate dimensions for the different components of container, and inclination of conical dome with horizontal, the capacity of water tank can be estimated in the following way.

Referring to the notations given in Fig 3, the capacity V of the container is given by:

Volume of liquid in the cylindrical portion of the tank (V_1) = $\frac{\pi}{4}D^2h$ (2.2)

Volume of liquid in the conical portion (V_2) = $\frac{\pi h_0}{12}\left[D^2 + D_0^2 + DD_0\right]$

$$V_2 = \frac{\pi \tan\phi_0 (D - D_0)}{24}\left[D^2 + D_0^2 + DD_0\right]$$

$$= \frac{\pi \tan\phi_0 (D - kD)}{24}\left[D^2 + k^2D^2 + kD^2\right]$$

$$V_2 = \frac{\pi \tan\varphi_0 D^3 (1 - k^3)}{24} \qquad (2.3)$$

The volume of the segment under the bottom spherical dome $(V_3) = \frac{h_2^2\pi}{3}\left[3R_2 - h_2\right]$

$$= \frac{h_2^2\pi}{3}\left\{\frac{3}{2}\left(\frac{R_0^2}{h_2} + h_2\right) - h_2\right\}$$

$$= \frac{h_2\pi}{6}\left[3R_0^2 + h_2^2\right]$$

$$= \frac{h_2\pi}{24}\left[3D_0^2 + 4h_2^2\right]$$

$$= \frac{k_0 D\pi}{24}\left[3k^2D^2 + 4k_0^2D^2\right]$$

$$V_3 = \frac{\pi k_0 D^3}{24}\left[3k^2 + 4k_0^2\right] \qquad (2.4)$$

The net capacity of tank $(V) = V_1 + V_2 - V_3$

$$= \frac{\pi}{4}D^2h + \frac{\pi \tan\phi_0 D^3 (1 - k^3)}{24} - \frac{\pi k_0 D^3}{24}\left[3k^2 + 4k_0^2\right]$$

$$V = \frac{\pi D^3}{24}\left[\frac{6h}{D} + \tan\varphi_0 (1 - k^3) - k_0(3k^2 + 4k_0^2)\right] \qquad (2.5)$$

The parameters, k, ϕ_0 and k_0 characterize the geometry of the tank as illustrated in Fig. 3.

2.2.3 According to U.P.R.N.N.L. [8]:

The ratio of height (h) and diameter (D) of cylindrical container shall be so fixed that height of container (h) is always equal to D/(3.5 to 4.5). The ratio of diameter (D) of cylindrical container to staging diameter (D_0) shall be so fixed that staging diameter is always equal to 0.68D to 0.78 D. The inclination of conical dome may be 45^0 to 50^0 with the horizontal. The rise of the domes should be about 1/5 to 1/8 times the diameter of the tank.

Referring to the notations given in Fig 4. the capacity V of the container is given by:

Volume of liquid in the cylindrical portion of the tank (V_1) = $\frac{\pi}{4} D^2 h$

$$= \frac{\pi k_1 D^3}{4} \qquad (2.6)$$

Volume of liquid in the conical portion (V_2) = $\frac{\pi h_0}{12}\left[D^2 + D_0^2 + DD_0\right]$

$$= \frac{\pi \tan\phi_0 (D - D_0)}{24}\left[D^2 + D_0^2 + DD_0\right]$$

$$= \frac{\pi \tan\phi_0 (D - k_2 D)}{24}\left[D^2 + k_2^2 D^2 + k_2 D^2\right]$$

$$V_2 = \frac{\pi \tan\varphi_0 D^3 (1 - k_2^3)}{24} \qquad (2.7)$$

The volume of the segment under the bottom spherical dome (V_3) = $\frac{h_2^2 \pi}{3}\left[3R_2 - h_2\right]$

$$= \frac{h_2^2 \pi}{3}\left\{\frac{3}{2}\left(\frac{R_0^2}{h_2} + h_2\right) - h_2\right\}$$

$$= \frac{h_2 \pi}{6}\left[3R_0^2 + h_2^2\right]$$

$$= \frac{h_2\pi}{24}\left[3D_0^2 + 4h_2^2\right]$$

$$V_3 = \frac{k_3 D\pi}{24}\left[3k_2^2 D^2 + 4k_3^2 D^2\right]$$

$$V_3 = \frac{\pi k_3 D^3}{24}\left[3k_2^2 + 4k_3^2\right] \tag{2.8}$$

The net capacity of tank (V) = $V_1 + V_2 - V_3$

$$= \frac{\pi k_1}{4} D^3 + \frac{\pi \tan\phi_0 D^3\left(1 - k_2^3\right)}{24} - \frac{\pi k_3 D^3}{24}\left[3k_2^2 + 4k_3^2\right]$$

$$V = \frac{\pi D^3}{24}\left[6k_1 + \tan\varphi_0\left(1 - k_2^3\right) - k_3\left(3k_2^2 + 4k_3^2\right)\right] \tag{2.9}$$

The parameters, k_1, k_2, k_3 and ϕ_0 characterize the geometry of the tank as illustrated in Fig. 4.

2.2.3 Suggestions of Dayaratnam [9]:

Dayaratnam suggested the approximate dimensions and reinforcement details for tanks of different capacities of thank. Table 2.gives the recommended dimensions of tank, Table 3 gives the details of top dome and ring beam, and Table 4 and 5 give the details of tank wall, middle ring beam and conical wall.

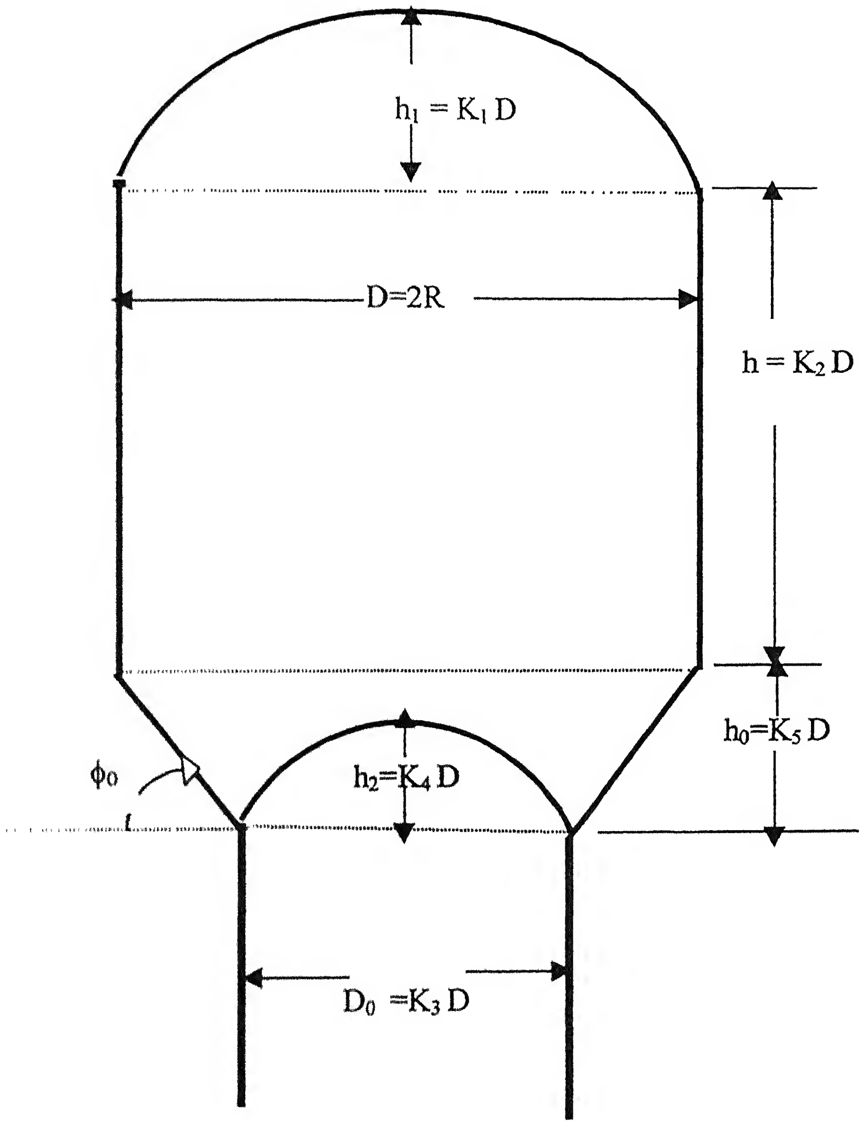


Fig. 2 Geometry of Intze tank [1]

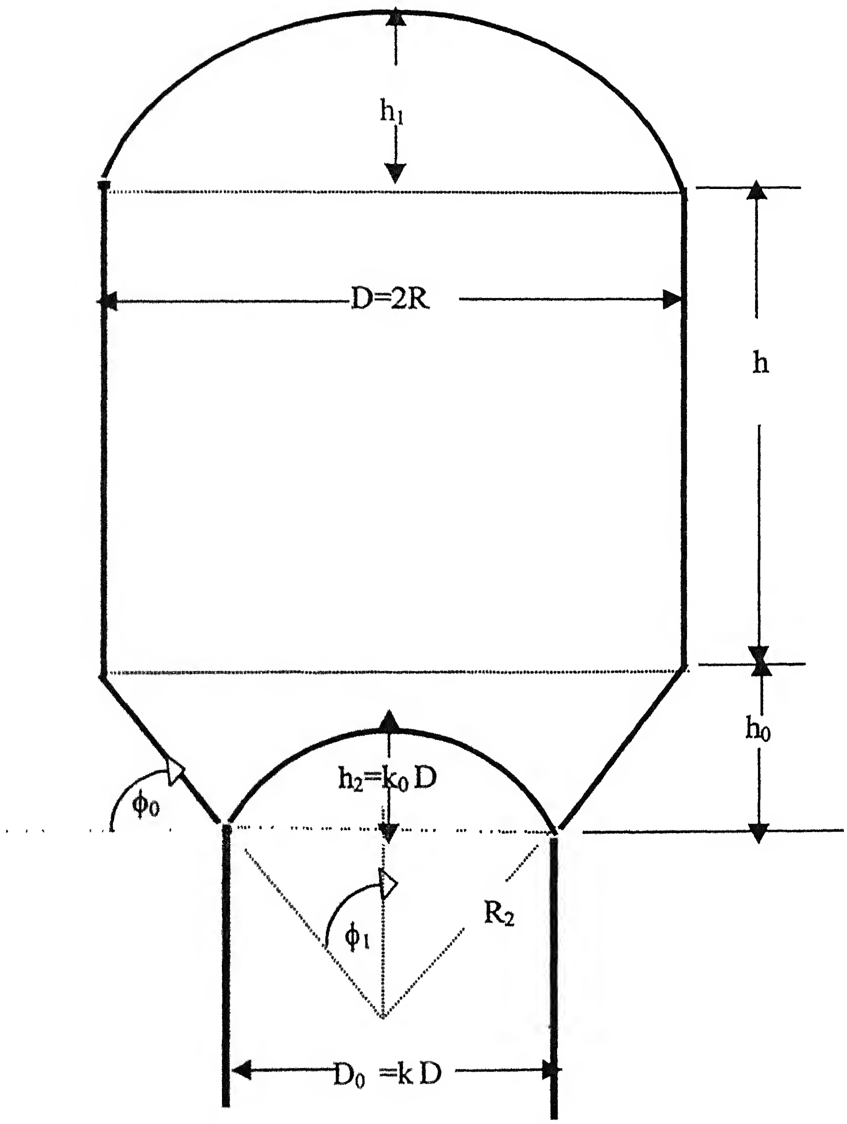


Fig. 3 Geometry of Intze tank [9]

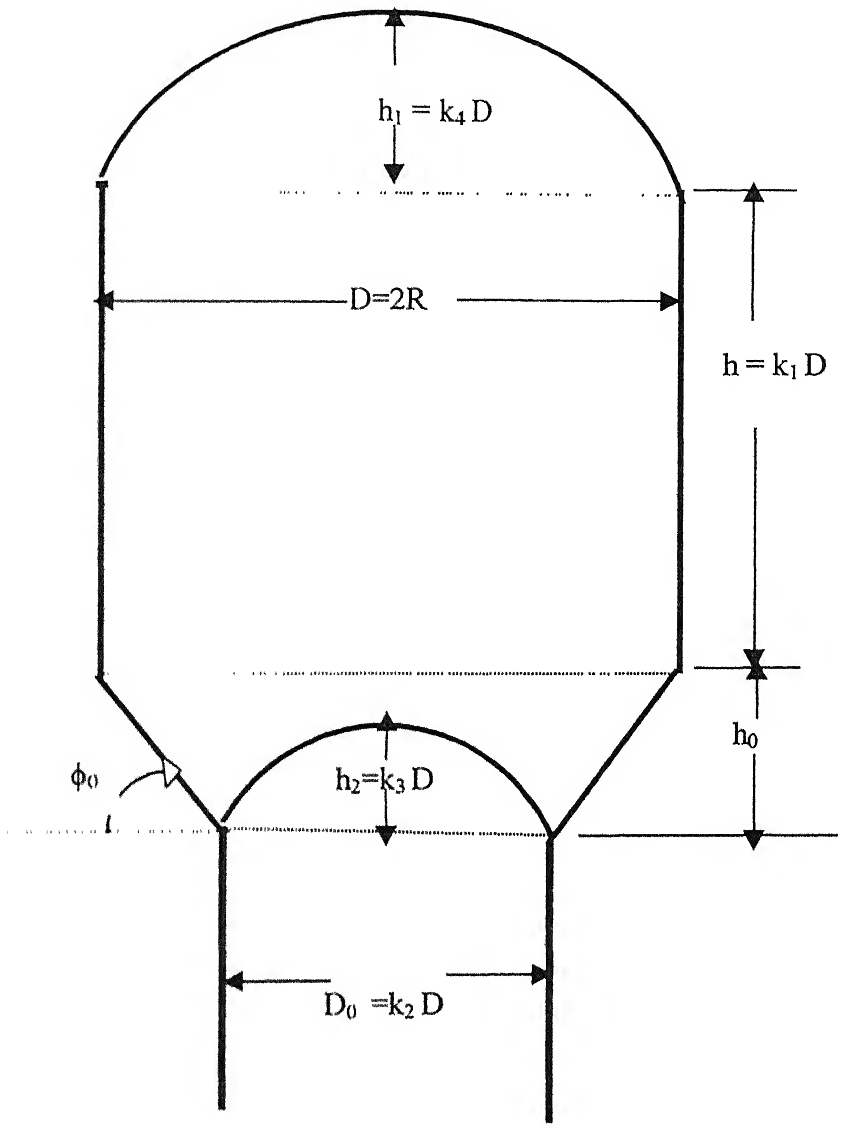


Fig. 4 Geometry of Intze tank [8]

Table 1. Geometrical parameters of container

	K_1	K_2	K_3	K_4	K_5
Billig [10]	0.125	0.60	0.625	0.125	0.188
Reynolds and Studman [11]	0.125	0.66	0.625	0.125	0.188
Jain et. al.[12]	0.125	0.30	0.70	0.14	0.179
Rao &Raghavan [13]	0.125	0.25	0.60	0.09	0.238

Table 2: Typical dimensions of intze tank (all dimensions in mm)

(For notations see Fig 2.)

No.	Capacity (Kl)	R	h	h_1	h_2	R_0	h_0
1	300	4500	4450	1205	1400	3375	1125
2	400	5000	4750	1340	1550	3750	1250
3	500	5500	4900	1475	1710	4125	1375
4	600	6000	4850	1600	1860	4500	1500
5	700	6500	4800	1740	2010	4875	1625
6	800	7000	4650	1875	2170	5250	1750
7	900	7250	4900	1940	2250	5440	1810
8	1000	7500	5100	2010	2330	5625	1875
9	1100	7750	5250	2080	2400	5810	1940
10	1200	7800	5350	2140	2480	6000	2000
11	1300	8250	5400	2210	2560	6190	2060
12	1400	8500	5500	2280	2640	6375	2125
13	1500	8750	5550	2340	2720	6560	2180
14	1600	9000	5550	2410	2800	6750	2250
15	1700	9250	5550	2480	2870	6940	2310
16	1800	9500	5550	2550	2870	7125	2375
17	1900	9750	5250	2610	3030	7310	2440
18	2000	10000	5550	2680	3100	7500	2500
19	2100	10250	5500	2750	3180	7690	2560

Table 2 continued….

Table 2 (continued)

20	2200	10450	5550	2800	3250	7840	2610
21	2300	10700	5500	2870	3320	8025	2675
22	2400	11000	5400	2950	3420	8250	2750
23	2500	11200	5450	3000	3480	8400	2800

Table 3: Structural details of top dome and ring beam (all dimensions are in mm)

		Top dome			Top ring beam					
No.	Capacity (Kl)	t	ϕ	s	b	ϕ	N	ϕ_v	s	Depth
1	300	80	8	225	170	12	4	8	200	350
2	400	80	8	225	190	12	4	8	200	380
3	500	80	8	225	210	14	4	8	200	420
4	600	80	8	225	230	14	4	8	200	460
5	700	80	8	225	250	16	4	8	200	480
6	800	90	8	225	265	16	4	8	200	535
7	900	90	8	225	265	18	4	8	200	535
8	1000	90	8	200	285	18	4	10	200	570
9	1100	90	8	200	305	18	4	10	200	610
10	1200	90	8	200	305	18	4	10	200	610
11	1300	90	8	200	315	20	4	10	200	630
12	1400	100	8	200	325	20	4	10	200	650
13	1500	100	8	200	325	20	4	10	200	650
14	1600	100	8	200	325	18	6	10	180	670

Table 3 continued….

Table 3 (continued)

15	1700	100	8	200	340	20	6	10	180	690
16	1800	100	8	200	340	22	6	10	180	690
17	1900	100	8	200	350	22	6	10	160	710
18	2000	100	8	200	360	25	6	10	160	725
19	2100	100	8	200	360	25	6	10	160	725
20	2200	100	8	200	375	25	6	10	160	740
21	2300	100	8	200	380	25	6	10	160	760
22	2400	100	8	200	380	25	6	10	160	760
23	2500	100	8	200	380	25	6	10	160	760

ϕ = Diameter of main bar, N = Number of main bars, ϕ_v= Diameter of ties,

s = spacing of reinforcement for single layer, t = thickness, b = width.

Table 4: Structural details of cylindrical wall, middle ring beam

(all dimension in mm)

		Detail of wall at bottom					Middle ring beam					
		Hoop			Vertical				Main		Stirrups	
No	Capacity (Kl)	t	ϕ	s	ϕ	s	b	Depth	ϕ	N	ϕ	s
1	300	155	16	110	10	130	550	150	20	4	8	150
2	400	185	16	100	10	130	700	150	20	4	8	150
3	500	210	16	100	10	130	825	150	20	4	8	150
4	600	225	16	100	10	130	995	150	22	4	8	150
5	700	240	18	100	10	125	1000	160	22	4	8	150

Table 4 continued….

Table 4 (continued)

6	800	250	18	100	10	125	1000	170	22	4	8	150
7	900	275	18	100	10	125	1000	195	22	6	8	150
8	1000	295	20	100	10	122	1000	215	22	6	10	200
9	1100	315	20	100	10	125	1000	240	22	6	10	200
10	1200	330	20	100	12	160	1000	265	25	6	10	200
11	1300	345	20	100	12	160	1000	285	25	6	10	200
12	1400	360	22	110	12	150	1000	305	28	6	10	200
13	1500	375	22	110	12	150	1000	325	28	6	10	200
14	1600	385	22	110	12	130	1000	345	28	6	12	300
15	1700	400	22	110	12	125	1000	365	28	6	12	300
16	1800	410	22	100	12	125	1000	385	28	6	12	300
17	1800	420	22	100	12	125	1000	385	28	6	12	300
18	2000	430	22	100	12	125	1000	405	28	8	12	300
19	2100	435	25	110	12	125	1100	405	28	8	12	300
20	2200	450	25	110	12	110	1100	425	28	10	12	300
21	2300	455	25	110	12	110	1100	445	28	10	12	300
22	2400	460	25	100	12	110	1150	440	28	10	12	300
23	2500	470	25	100	12	110	1150	460	28	10	12	300

Table 5 Structural details of bottom dome and conical wall (all dimension in mm)

		Bottom dome			Conical dome				
					Hoop			Inclined	
No	Capacity (Kl)	t	ϕ	s	t	ϕ	s	ϕ	s
1	300	150	8	150	215	22	120	12	160
2	400	150	8	150	255	22	120	12	160
3	500	160	8	125	290	22	120	12	160
4	600	160	8	125	315	22	120	12	160
5	700	160	8	125	335	25	140	12	125
6	800	160	10	200	350	25	140	12	125
7	900	160	10	200	380	25	110	12	125
8	1000	180	10	200	410	28	110	12	125
9	1100	180	10	200	435	28	110	12	125
10	1200	200	10	200	460	28	120	12	125
11	1300	200	10	200	480	28	120	12	125
12	1400	200	10	200	500	28	120	12	125
13	1500	225	12	225	520	28	110	12	125
14	1600	225	12	225	535	28	110	12	125
15	1700	225	12	225	550	28	110	12	125
16	1800	225	12	225	565	28	110	12	125

Table 5 continued....

Table 5 (continued)

17	1800	225	12	225	580	28	100	12	125
18	2000	225	12	225	590	28	100	12	125
19	2100	250	12	225	605	28	100	12	125
20	2200	250	12	225	620	28	100	12	125
21	2300	250	12	225	630	28	90	12	125
22	2400	250	12	225	635	28	90	12	125
23	2500	250	12	225	650	28	90	12	125

The radii (R_1, R_2) and the corresponding dome –angle (ϕ, ϕ_1) of the top and bottom domes (Fig. 5) are given as:

$$\text{Radius of top dome } (R_1) = \frac{1}{2}\left(\frac{R^2}{h_1} + h_1\right) \tag{2.10}$$

$$\sin\phi = R/R_1,$$

$$\text{Radius of bottom dome } (R_2) = \frac{1}{2}\left(\frac{R_0^2}{h_2} + h_2\right) \tag{2.11}$$

$$\sin\phi_1 = R_0/R_2$$

2.3 Preliminary Selection of Staging Configurations:

The number of columns to be adopted is decided based on the column spacing, which normally lies between 3.6 and 4.5 m [14]. For a given staging height, the height of each panel lies between 3.5 to 4.5 m [8]. The shape of the columns are generally taken as circular. The size of the column lies between 400mm to 600mm in diameter and the size of bracing lies between 200mm×450mm to 250mm × 750mm [8].

2.4 Preliminary Design of Foundation:

In order to obtain rigidity at the column base, combined footing is generally provided for intze tank. Depending upon the allowable soil bearing pressure, the combined footing maybe either in the form of a solid circular raft or an annular circular raft. For determining the approximate area of foundation, the value of total vertical load on foundation may be assumed as 140 to 150 percent of weight of water [8]. Depths of foundation depend upon the property of soil. For the earthquake condition the value of soil being capacity is generally increased by 50 percent [15].

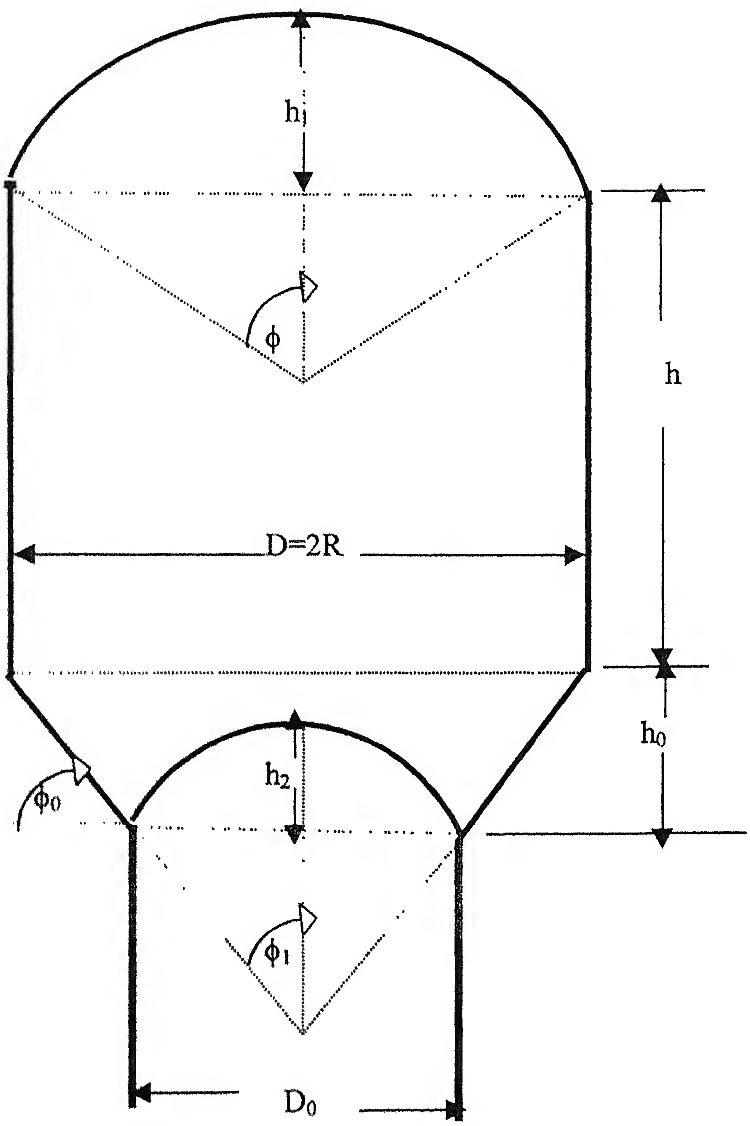


Fig. 5 Geometry of Intze tank

Chapter 3

Analysis and Design of Water Tank

3.1 Introduction

After the completion of the preliminary design, the next task is to perform the complete and thorough analysis and design of the entire tank structure following a suitable sequence. Presented in the following, is the currently adopted methods of design of different components of the structure based on well established approaches.

The container is designed for uncracked condition using working stress method. In limit states method, generally, the concrete section is assumed to be cracked, and, hence, the limit states method is not suitable for the container design.

3.2 Analysis and Design of the Container

The Intze tank consists of four shells, i.e. two spherical, one conical and one cylindrical shell, which are connected by three ring beams as shown in Fig.1. The gravity loads and the associated forces on each shell are symmetrical about the axis of revolution, and, pure membrane state of stress would exist so long as each shell is simply supported at its edge, i.e. it is able to undergo resulting edge displacements without restraint, while the supports supply the necessary reaction to balance the meridional forces. But in the Intze tank, at the junction of shells, the edge displacements are actually restrained. Thus the complete analysis requires both membrane and continuity analysis.

3.2.1 Membrane Analysis and Design:

3.2.1.1 Top spherical dome: The top dome can be designed for the meridional thrust at the edge and for maximum hoop force as shown in Fig. 6.

Assuming thickens of dome = t mm

$$\text{Total force P per sqr m of dome} = t \times \gamma_c + L.L \quad (3.1)$$

Where γ_c = unit weight of concrete

L. L. = live load (including snow load)

$$\text{Meridional thrust at edge } (T_1) = \frac{PR_1}{1+\cos\phi} \quad (3.2)$$

$$\text{Meridional stress } (\sigma_{md}) = T_1/t$$

$$\text{Maximum hoop force } (T_2) = \frac{PR_1(\cos^2\phi + \cos\phi - 1)}{1+\cos\phi} \text{ at } \phi=0 \quad (3.3)$$

$$\text{Maximum hoop stress } (\sigma_{hd}) = T_2/t$$

If the stresses are within the safe limits, minimum reinforcements are to be provided. According to the relevant Indian Standard Code [14], the minimum reinforcement in wall, floors and roofs in each of the two directions at right angles, shall have an area of 0.3 percent of the concrete section in that direction for section up to 100 mm thick. For section of thickness greater than 100 mm and less than 450 mm, the minimum reinforcement in each of the two directions shall be linearly reduced from 0.3 percent for 100 mm thick section to 0.2 percent for 450 mm thick section. For section of thickness greater than 450 mm, minimum reinforcement in each of the two directions, shall be kept

at 0.2 percent. In concrete section of thickness 225 mm or greater, two layers of reinforcing steel shall be placed one near each face of the section to make up the minimum reinforcement.

For example, for top roof dome thickness up to 100 mm:

Area of minimum reinforcement (for stress less than the permissible stress of concrete)=

$$A_{s1} = \frac{0.3 \times t \times 1}{100} \text{ per unit length} \tag{3.4}$$

Spacing of reinforcement c/c in both directions $(S_1) = \dfrac{1000 \times A_{\varphi}}{A_{s1}}$ (3.5)

Where A_{ϕ} = Cross-sectional area of each bar (generally using 8 mm ϕ bars)

3.2.1.2 Top Ring Beam B_1: The meridional thrust, T_1, exerts a vertical load $T_1 \sin\phi$ on the wall and also it imposes an outward radial force $T_1 \cos\phi$ which cause hoop tension in the ring beam B_1 as shown in Fig. 7.

Radial outward reaction from top dome $(P_1) = T_1 \cos\phi$ N/m

Total hoop tension tending to rupture the beam= $P_1 \times D/2$

Area of reinforcement in beam $(A_{s2}) = \dfrac{P_1 \times D}{\sigma_{st} \times 2}$ (3.6)

Where σ_{st}= permissible stress in high yield strength deformed bars.

Number of bars $(N_2) = A_{s2} / A_{\phi 1}$

Where $A_{\phi 1}$ = Cross-section area of each bars.

The area of cross-section of ring beam B_1

$$A_{b1} = \frac{P_1 \times D}{\sigma_{tc} \times 2} - (m-1) A_{s2} \tag{3.7}$$

$A_{b1} = b_1 \times d_1$

Where σ_{tc} = Allowable tensile stress of concrete

m = modular ratio

b_1 and d_1 are width and depth of ring beam B_1

3.2.1.3 Cylindrical tank wall: The wall is assumed free to deform at both edge and is thus under a pure hoop tension. The maximum hoop tension can be found at the bottom and the wall can be designed to resist this hoop tension. As the pressure varies linearly with depth, hoop tension will also vary linearly as shown in Fig. 8.
Hoop tension at any depth = P_h D/2 , where P_h is the pressure at depth h.

According to the relevant Indian Standard Code [15] the pressure on the wall due to earthquake (i.e. hydrodynamic pressure) given by:

$$p_{wh} = \alpha_h \, wh \sqrt{3} \cos \phi' \left[\frac{y}{h} - \frac{1}{2}\left(\frac{y}{h}\right)^2 \right] \tanh \sqrt{3}\left(\frac{R}{h}\right) \qquad (3.8)$$

Where h, y, R and ϕ' are as defined in Fig. 9.

It will be maximum at the base of the wall corresponding to the values of ϕ' and y being equal to zero and h, respectively.

Consideration in design is made for the following two different cases:

a) **For Earthquake condition**

Hoop reinforcement:

Maximum hoop tension (at the base of the wall) (P_2) is given by:

$$P_2 = \frac{whD}{2} + \left(p_{wh}\right)_{at\,\varphi'=0\,and\,y=h} \times \frac{D}{2} \qquad (3.9)$$

Area of steel at the base of wall (A_{s3}) = $P_2/(1.33\ \sigma_{st})$ (3.10)

Providing horizontal reinforcements on both the faces, area of reinforcement on each face is $A_{s3}/2$.

$$\text{Spacing of horizontal reinforcements at bottom } (S_2) = \frac{1000 \times A_{\varphi 2} \times 2}{A_{s3}} \qquad (3.11)$$

Since the hoop tension is to be approximate varies linearly then the area of reinforcement and spacing also varies from bottom to top.

Distribution reinforcement:

$$\text{Maximum thickness of the wall } (t_{wmax}) = \frac{1}{1000}\left(\frac{P_2}{1.33\sigma_{tc}} - (m-1)A_{s3}\right) \qquad (3.12)$$

The minimum thickness ($t_{w\,min}$) of wall is to be assuming by the help of the preliminary design.

$$\text{Average thickness of wall } (t_{w\,ave}). = (t_{w\,max} + t_{w\,min})/2 \qquad (3.13)$$

$$\text{Percentage of steel distribution } (r) = 0.3 - \left[\frac{t_{wave} - 100}{t_{wmax} - t_{wave}}\right] \qquad (3.14)$$

$$\text{Area of distributions reinforcement } (A_{s4}) = \frac{r \times t_{wave} \times 1000}{100}\, mm^2 \qquad (3.15)$$

Area of reinforcement on each face = $A_{s5}/2$

$$\text{Spacing of distribution reinforcement } (S_3) = 2(1000 \times A_{\phi 3})/A_{s4} \qquad (3.16)$$

Where $A_{\phi 3}$ is area of reinforcement.

b) **Without considering Earthquake condition:**

Hoop reinforcement:

$$\text{Maximum hoop tension (at the base of the wall) } P^1{}_2 = w\,h\,D/2 \qquad (3.17)$$

$$\text{Area of steel at the base of wall } (A^1{}_{s3}) = P^1{}_2/(\sigma_{st}) \qquad (3.18)$$

Providing horizontal reinforcements on both the faces, area of reinforcement on each face is $A^1{}_{s3}/2$.

$$\text{Spacing of horizontal reinforcements at bottom } (S_3^1) = \frac{1000 \times A_{\varphi 2}}{A_{s3}^1} \tag{3.19}$$

Since the hoop tension is to be approximate varies linearly then the area of reinforcement and spacing also varies from bottom to top.

Distribution reinforcement:

$$\text{Maximum thickness of the wall } (t^1_{w\max}) = \frac{1}{1000}\left(\frac{P_2^1}{\sigma_{tc}} - (m-1)A^1{}_{s3}\right) \tag{3.20}$$

The Minimum thickness ($t_{w\,min}$) of wall is to be assuming by the help of the preliminary design.

Average thickness of wall ($t^1{}_{w\,avg}$).= ($t^1{}_{w\,max}$+ $t_{w\,min}$)/2

$$\text{Percentage of steel distribution } (r^1) = 0.3 - \left[\frac{t^1\text{wave} - 100}{t_{\text{wmax}} - t_{\text{wmin}}}\right] \tag{3.21}$$

$$\text{Area of distributions reinforcement } (A^1{}_{s4}) = \frac{r^1 x t^1{}_{wave} \times 1000}{100} mm^2 \tag{3.22}$$

Area of reinforcement on each face = $A^1{}_{s4}/2$

Spacing of distribution reinforcement ($S^1{}_3$) =2(1000× $A_{\phi 3}$)/ $A^1{}_{s4}$ (3.23)

Where $A_{\phi 3}$ is cross-section area of reinforcement.

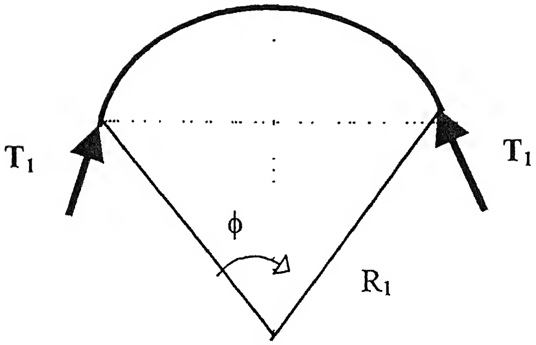


Fig. 6 Forces on top dome

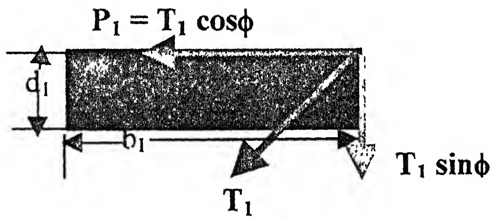


Fig. 7 Forces on upper ring beam (B_1)

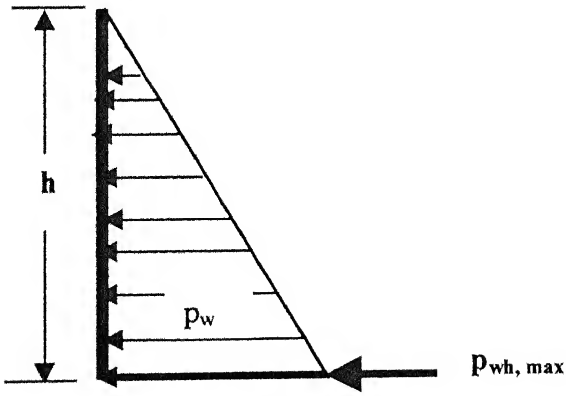
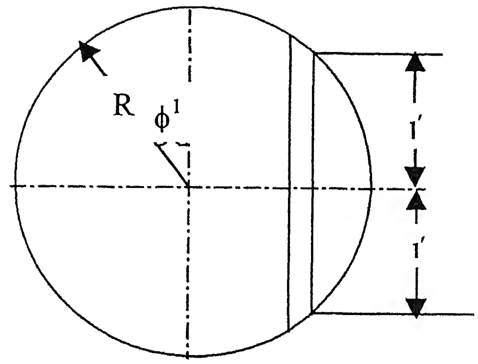
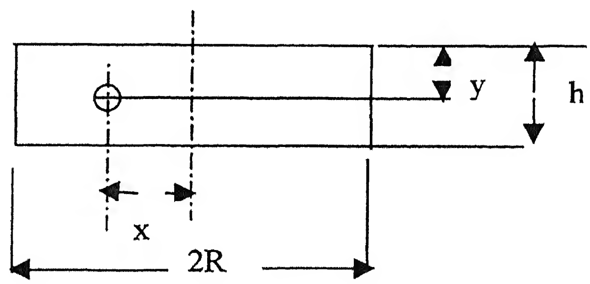


Fig. 8 Forces on cylindrical wall



Circular tank (Plane)



Circular Tank (Elevation)

Fig.9 Plane and Elevation of Cylindrical tank wall

3.2.1.4 Ring beam B_3: This Ring Beam is designed to resist the hoop tension caused by vertical load (due to weight of top dome, upper ring beam, cylindrical wall and self weight of middle ring beam B_3) at the junction of wall with conical dome as shown in Fig. 10.

$$\text{Load of top dome } (W_1) = T_1 \sin\phi \quad (3.24)$$

$$\text{Load due to ring beam } B_1\ (W_2) = d_1\ (b_1 - t_{wave}) \times 1 \times \gamma_c \quad (3.25)$$

$$\text{Load due to tank wall } (W_3) = h \times t_{wave} \times 1 \times \gamma_c \quad (3.26)$$

$$\text{Self weight of beam } B_3\ (W_4) = (b_3 - t_{w\,max}) \times 1 \times \gamma_c \quad (3.27)$$

$$\text{Total load on beam } (W_5) = W_1 + W_2 + W_3 + W_4 \quad (3.28)$$

$$\text{Hydrostatic pressure on the beam } (P_{b3}) = w\ h\ d_3 \quad (3.29)$$

$$\text{Hydrodynamic presser on the beam } P_{hd,b} = (p_{wh})_{at\ \phi' = 0 \text{ and } y = h} \times b_3 \quad (3.30)$$

Consideration in design is made for the following two different cases:

a) **For Earthquake condition**

$$\text{Hoop tension in the ring beam } (P_3) = (W_5 \tan\phi_0 + P_{b3} + P_{hd,b}) \times D/2 \quad (3.31)$$

Where ϕ_0 is the inclination of conical dome with vertical.

$$\text{Area of hoop reinforcement } (A_{s5}) = P_3/1.33\sigma_{st} \quad (3.32)$$

$$\text{Number of bars } (N_3) = A_{s6}/A_{\phi 4}$$

$$\text{Tensile stress in equivalent section } (\sigma_{tc}) = P_3/\{b_3 \times d_3 + (m-1)\ A_{s5}\} \quad (3.33)$$

b) **Without considering Earthquake condition:**

$$\text{Hoop tension in the ring beam } (P^1{}_3) = (W_5 \tan\phi_0 + P_{b3}) \times D/2 \quad (3.34)$$

Where ϕ_0 is the inclination of conical dome with vertical.

$$\text{Area of hoop reinforcement } (A^1{}_{s5}) = P^1{}_3/\sigma_{st} \quad (3.36)$$

$$\text{Number of bars } (N_3) = A^1{}_{s5}/A_{\phi 4}$$

Tensile stress in equivalent section (σ_{tc}) = $P^1_3 / \{b_3 \times d_3 + (m-1) A^1_{s5}\}$

3.2.1.5 Conical dome: The conical dome supports a uniform vertical load from walls at its top edge. At top of the dome a hoop tension is created which exerts a radial inward force at each slanting strip (dome is assumed as consisting of individual slanting strips monolithically joined) and opposes its rotation outward. The magnitude of the radial force created at top edge is so much that combining with the vertical load, the resultant lies along the meridian of conical dome. Thus the vertical load at top edge of the conical dome is supported by it with the creation of meridional thrust and a hoop tension. The water pressure on conical dome and its own weight acting at any point give rise to hoop tension at each plane, whose inward reaction, together with water pressure and weight of dome, cause a resultant force which is meridional. All the forces are shown in Fig. 11.

3.2.1.5.1 Meridional thrust:

$$\text{Weight of conical dome } (W_6) = \left[\pi\left(\frac{D+D_0}{2}\right) \times l \times t_{con}\right]\gamma_c \tag{3.37}$$

where, $l = \sqrt{(R-R_0)^2 + h_0^2}$

Weight of water resting directly on conical dome

$$W_7 = \gamma_w \frac{\pi}{4}\left(D^2 - D_0^2\right) \times h + \frac{\gamma_w \pi h_0}{12}\left(D^2 + D_0^2 + DD_0\right) - \frac{\pi}{4} \times D_0^2 \times h \times \gamma_w \tag{3.38}$$

Total weight above beam (B_2) $W_8 = W_5 \times \pi D + W_6 + W_7$ (3.39)

Vertical load per meter $W_9 = W_8 / \pi D_0$ (3.40)

Minimum meridional thrust at conical dome (T_3)= $W_9 / \cos\phi_0$ (3.41)

Meridional stress (σ_{mcon}) $= T_3 / t_{con}$

3.2.1.5.2 Hoop tension: Due to water pressure and self-weight, the conical dome is subjected to hoop tension.

Diameter of conical dome at height h^1 from base $D^1 = D_0 + (D - D_0)\, h^1/2$ (3.42)

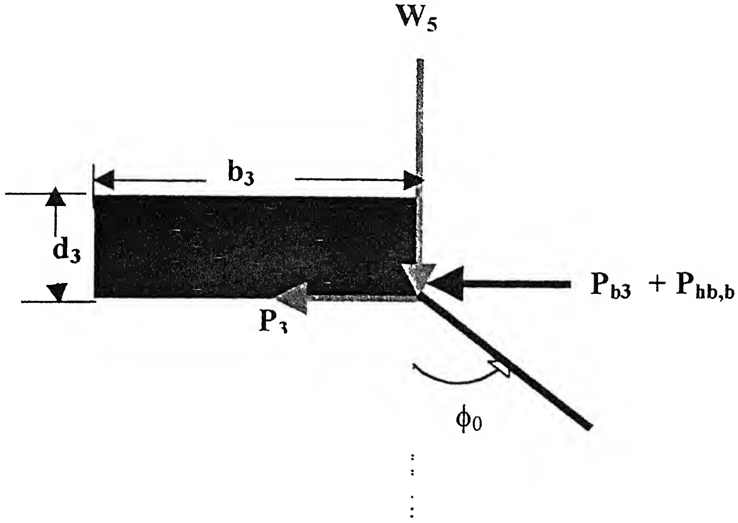


Fig. 10 Forces on middle ring beam (B_3)

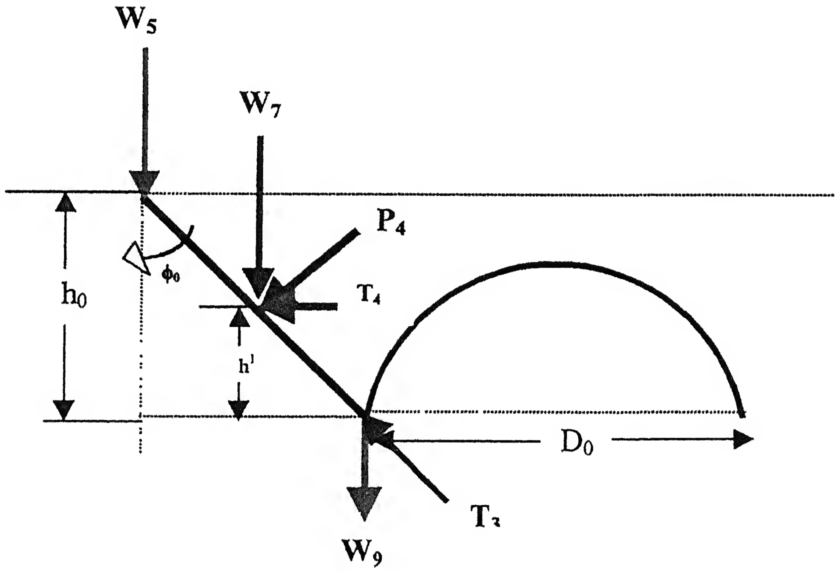


Fig. 11 Forces on conical dome

Self weight of conical dome per meter square $(W_{10}) = t_{con} \times 1 \times 1 \times \gamma_w$ (3.43)

Intensity of water pressure at height h^1 from the base $P_4 = (h + h_0 - h^1) \times \gamma_w$ (3.44)

Hoop tension in the ring at height h^1 above base

$$T_4 = \left(\frac{P_4}{\cos\phi_0} + W_{10} \tan\phi_0 \right) \times \frac{D^1}{2} \quad (3.45)$$

Hoop tension in the ring will be calculated at height $h^1=0$ to $h^1=h_0$

For maximum value of hoop tension in the ring $dT_4/dh^1 = 0$, (3.46)

From which, $h^1 = \dfrac{W_{10} \sin\phi_o (D - D_0)}{2\gamma_w} + \dfrac{(h + h_o)(D - D_0)}{2} - D_o$

3.2.1.5.3 Design of wall:

Meridional stress = σ_{mcon}

Maximum hoop tension in the ring $= T_{4\,max}$

Area of hoop reinforcement $(A_{s6}) = T_{4\,max} / \sigma_{st}$ (3.47)

Area of hoop reinforcement in each face $= A_{s6}/2$

Spacing of bars $(S_4) = (1000 \times A_{\phi 5}) \times 2 / A_{s6}$ (3.48)

Maximum tensile stress in composite section $(\sigma_{ct}) = \dfrac{T_{4max}}{(t_{con} \times 1000) + (m-1)A_{s6}}$ (3.49)

σ_{ct} should be less than the permissible value.

If σ_{ct} is greater than the permissible value then the thickness of conical dome is to be increase.

Percentage reinforcement in meridional direction

$$r_1 = 0.3 - \left[\frac{t_{cmax} - 100}{t_{cmax} - t_{cmin}} \right] \times 0.1 \qquad (3.50)$$

Area of reinforcement in meridional direction (A_{s7}) =($r_1 \times t_c \times 1000$)/100 (3.51)

Area of meridional reinforcement on each face = $A_{s7}/2$

Spacing of meridional reinforcement (S_5) =2(1000 $\times A_{\phi6}$)/A_{s7} (3.52)

3.2.1.6 Bottom dome: Like the top spherical dome, the bottom dome also develops only compressive stresses both meridionally and along hoops as shown in Fig. 12.

Weight of water above the surface of the dome

$$W_{11} = \left[\frac{\pi D_0^2 H_0}{4} - \frac{\pi h_2^2}{3}(3R_2 - h_2) \right] .w \qquad (3.53)$$

Where H_0 is the total depth of water above the edge of the dome.

Self-weight of the dome (W_{12}) = $2\pi \times R2 \times h2 \times t_{bd} \times \gamma_c$ (3.54)

Total load (W_{13}) = $W_{11}+W_{12}$ (3.55)

Meridional thrust (T_5) = $W_{13}/(\pi D_0 \sin\phi_1)$ (3.56)

Meridional stress (σ_{mbd}) = T_5/t_{bd}

Intensity of load per unit area (W_{14}) = $W_{13}/(2\pi R_2 h_2)$ (3.57)

Maximum hoop stress at center (σ_{hbd}) = $W_{14} R_2/2t_{bd}$

If the hoop and meridional stresses are within allowable limits, minimum reinforcements are to be provided.

Area of minimum percentage reinforcement $r_2 = 0.3 - \left[\frac{t_{bd} - 100}{450 - 100} \right] \times 0.1$ (3.58)

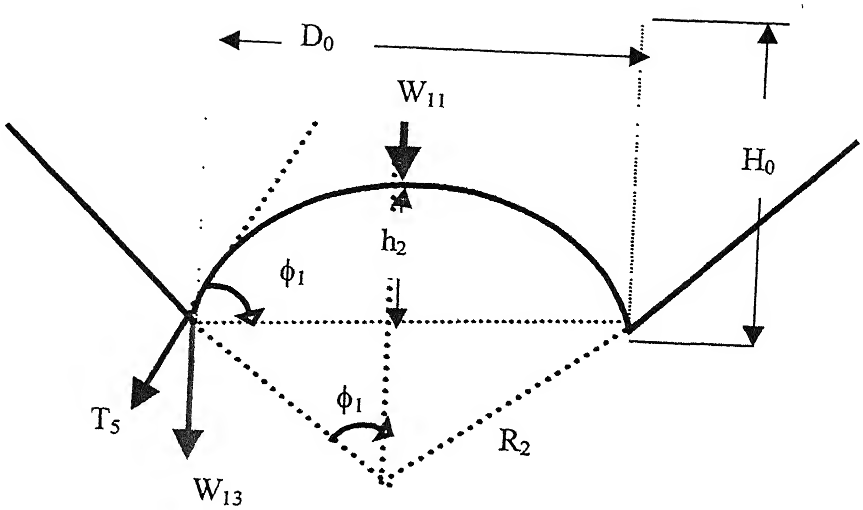


Fig. 12 Forces on bottom dome

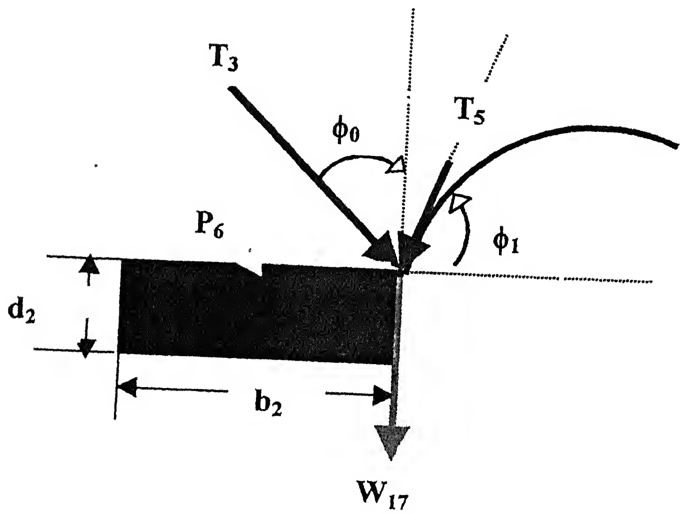


Fig. 13 Forces on bottom ring beam (B_2)

Area of minimum reinforcement $(A_{s8}) = (r_2 \times b_{bd} \times d_{bd})/100$, in each direction (meridional as well as hoop).

Spacing of bars in each direction $(S_6) = (1000 \times A_{\phi 7})/A_{s8}$

3.3.1.7 Ring Beam B_2: This beam receives an inward inclination thrust from the conical dome and inclination outward thrust due to the reaction from the bottom spherical dome as shown in Fig.13. The vertical components of these two thrusts add up and the ring beam is design for this load. Their horizontal components oppose each other and depending upon their relative magnitudes, the ring beam is subjected to either hoop tension or hoop compression, and, can be designed accordingly.

$$\text{Horizontal force } (P_5) = T_3 \sin\phi_0 - T_5 \cos\phi_1 \tag{3.59}$$

$$\text{Hoop stress in beam } (\sigma_{hbb}) = (P_5 D_0)/2\, b_2 d_2 \tag{3.60}$$

$$\text{Vertical load per unit length } (W_{15}) = T_3 \cos\phi_0 + T_5 \sin\phi_1 \tag{3.61}$$

$$\text{Self weight of beam } B_2 \ (W_{16}) = b_2 \times d_2 \times 1 \times \gamma_c \tag{3.62}$$

$$\text{Total load on beam } (W_{17}) = W_{15} + W_{16} \tag{3.63}$$

$$\text{Maximum negative bending moment at support } (M_0) = C_1 W_{17} R^2_0\, 2\theta \tag{3.64}$$

$$\text{Maximum positive bending moment } (M_c) = C_2 W_{17} R^2_0\, 2\theta \tag{3.65}$$

$$\text{Maximum torsional moment } (M^t_\alpha) = C_3 W_{17} R^2_0\, 2\theta \tag{3.66}$$

Where C_1, C_2, C_3 are the coefficient of bending and twisting moment

$2\theta = 2\pi/n$, where n is the number of columns.

$$\text{Required depth of beam } d_2 = \sqrt{\frac{M}{R^1 b_2}} \tag{3.67}$$

Where M is the maximum bending moment.

$$\text{Maximum shear force at support } (F_0) = W_{17} R_0\, \theta \tag{3.68}$$

$$\text{Shear force at any point } (F) = W_{17} R_0 (\theta - \alpha) \tag{3.69}$$

$$\text{Shear force at the point of maximum torsional moment } (F_m) = W_{17} R_0 (\theta - \alpha_m) \tag{3.70}$$

Bending moment at the point of maximum torsional moment

$$M\alpha_m \text{ (sagging)} = W_{17} R^2_0 (\theta \sin\alpha_m + \theta \operatorname{Cot}\theta \operatorname{Cos}\alpha_m - 1) \tag{3.71}$$

The torsional moment at any point

$$M^t\alpha = W_{17} R^2_0 \{\theta \operatorname{Cos}\alpha - \theta \operatorname{Cot}\theta \operatorname{Sin}\alpha - (\theta - \alpha)\} \tag{3.72}$$

At the support $\alpha = 0$, $M^t_0 = 0$

At the mid-span $\alpha = \theta$, $M^t_c = 0$

Hence the following combinations of bending moment and torsional moment are obtained:

(a)At the point of maximum torsion:

Bending moment $= M\alpha$ and Torsional moment $= M^t_{\alpha m}$

(b) At the support: Bending moment $= M_0$ and Torsional moment $= M^t_0$

(c) At mid span : Bending moment $= M_c$ and Torsional moment $= M^t_c$

The c/s area required for the main reinforcement is obtained with the consideration of the above three conditions.

(i) Main Reinforcement

(a) Section at point of maximum torsion

$T = M^t_{max}$

$M_\alpha = M$

$$\text{Equivalent bending moment } (M_{e1}) = M + T\left[\frac{1+\frac{d_2}{b_2}}{1.7}\right] \tag{3.73}$$

$$\text{Area of steel } (A_{s9}) = M_{e1}/(\sigma_{st}\, j\, d_{2f}) \tag{3.74}$$

$$\text{Numbers of bars } (N_4) = A_{s9}/A_{\phi 7}$$

If T > M, the compression reinforcements are to be provided.

$$\text{Equivalent bending moment}\,(M_{e2}) = T\left[\frac{1+\frac{d_2}{b_2}}{1.7}\right] - M \tag{3.75}$$

$$\text{Compression reinforcement } (A_{s10}) = M_{e2}/(\sigma_{st}\, j\, d_{2f}) \tag{3.76}$$

$$\text{Numbers of bars } (N_5) = A_{s10}/A_{\phi 7}$$

(b) Section at maximum hogging bending moment (support):

$M_0 = M_{max}$, $M^t_0 = 0$

$$\text{Area of steel } (A_{s11}) = M_0/(\sigma_{st}\, j\, d_{2f}) \tag{3.77}$$

$$\text{Numbers of bars } (N_6) = A_{s11}/A_{\phi 7}$$

(c) Section at maximum sagging bending moment (mid-span):

Maximum bending moment at center= M_c

Torsional moment (M^t_c) = 0

Area of steel (A_{s12}) = $M_c/(\sigma_{st}\, j\, d_{2f})$, Numbers of bars ($N_7$) = $A_{s12}/A_{\phi 7}$ (3.78)

(ii) Transverse reinforcement:

(a) At point of maximum torsional moment:

At the point of maximum torsion shear force (V) = F_m

$$\text{Equivalent shear force } (V_e) = V + 1.6\, T/b_2 \tag{3.79}$$

$$\text{Shear stress } (\tau_{ve}) = V_e/b_2\, d_2 \tag{3.80}$$

τ_{ve} should be always less than $\tau_{c\,max}$ otherwise the section is to be redesign [16].

If $\tau_{ve} > \tau_c$, shear reinforcement is necessary, otherwise minimum shear reinforcements are to be provided [16].

Cross section area of the stirrups [17]

$$A_{sv} = \frac{T \times S_{v}}{b^1 d^1 \sigma_{sv}} + \frac{V \times S_v}{2.5 d^1 \sigma_{sv}} \tag{3.81}$$

Where b^1 = center- to- center distance between corner bars in the direction of the width.

d^1 = center- to- center distance between corner bars in the direction of depth.

But the total reinforcement shall not be less than $(\tau_{ve} - \tau_c)\, b_2\, S_v/\sigma_{sv}$

The spacing of the stirrups shall not exceed the least of x_1, $(x_1+y_1)/4$, and 300 mm, where x_1 and y_1 are the respectively the shorter and long dimension of the stirrup.

(b) At the point of maximum shear (supports):

Shear stress $\tau_v = F_0 / b_2 d_2$

τ_v should be always less than $\tau_{c\,max}$, otherwise the section is to be redesigned [16]

If $\tau_v > \tau_c$, then shear reinforcement is necessary, otherwise minimum shear reinforcements are to be provided [16].

$$V_c = \tau_c \times b_2 d_2$$

Design shear force $(V_s) = F_0 - V_c$ (3.82)

The spacing of stirrups $(S_{v1}) = (\sigma_{sv} \times A_{sv1} \times d_2)/V_s$ (3.83)

Where A_{sv1}= area of 4-leggd stirrups.

(c)At mid-span: At the mid-span, shear force is zero. Hence minimum/nominal reinforcement are to be provided.

Minimum shear reinforcement in the form of stirrups shall be provided such that [16]

$$\frac{A_{sv2}}{b_2 S_{v2}} \geq \frac{0.4}{0.87 f_y} \tag{3.84}$$

Where A_{sv2} = total cross-section area of stirrups legs effective in shear,

S_{v2}= stirrups spacing along the length of the member,

b_2= breadth of the beam or breadth of the web of flanged beam, and

f_y = characteristic strength of the stirrup reinforcement in N/mm^2 which shall not be taken greater than 415 N/mm^2.

Maximum spacing of shear reinforcement measured along the axis of the member shall not exceed 0.75 d. In no case shall the spacing exceed 300mm.

(iii) Side face reinforcement: where the depth of the web in a beam exceeds 450 mm, side face reinforcement shall be provided along the two faces. The total area of such reinforcement shall be not less than 0.2 percent of the web area and shall be distributed equally on two faces at spacing not exceeding 300mm or web thickness whichever is less [16].

$$A_{sd} = (0.2 \times b_2 \times d_2)/100 \text{ mm} \tag{3.85}$$

3.2.2 Continuity analysis:

The forces due to continuity are obtained by applying the principle of consistent deformations. The vertical displacements are always consistent as each shell is free to deform in this direction, consistency has only to be satisfied for horizontal and angular displacements between the shells meeting at a joint. Based on the stiffness of each shell at edges for horizontal and angular moments, equations of consistency are framed and the forces due to continuity can be determined after solving this system of simultaneous equations in rotation (ψ) and displacement (δ).

3.2.2.1 Effect of continuity at the junction of top-dome, ring beam (B_1) and cylindrical portion:

3.2.2.1.1 Membrane deformation and stiffness [7]: In order to determine the effect of continuity of joints, first the membrane deformation and stiffness at each edge are to be calculated.

(i) Top dome: (Fig 14 and 15)

$$\text{The slope at edge } (\psi_d) = \frac{2PR_1 Sin\phi}{Et} radian \tag{3.86}$$

$$\text{T he horizontal outward deflection } (\delta_d) = \frac{PR_1^2 Sin\phi}{Et}\left[\frac{1}{1+Cos\phi} - Cos\phi\right] \tag{3.87}$$

$$\text{Moment stiffness per unit length of periphery at edge } (M_1) = \frac{ER_1 t}{4\alpha_1^3}\left[K_1 + \frac{1}{K_1}\right] \tag{3.88}$$

$$\text{Corresponding radial force per unit length } (H_1) = \frac{Et}{2\alpha_1^2 K_1 Sin\phi} \tag{3.89}$$

$$\text{Thrust stiffness per unit length } (H_2) = \frac{Et}{\alpha_1 R_1 K_1 Sin^2\phi} \tag{3.90}$$

$$\text{Corresponding moment per unit length } (M_2) = \frac{Et}{2\alpha_1^2 K_1 Sin\phi} \tag{3.91}$$

Where, $\alpha_1^4 = 3\left(\frac{R_1}{t}\right)^2$

$$K_1 = 1 - \frac{1}{2\alpha_1} Cot\phi$$

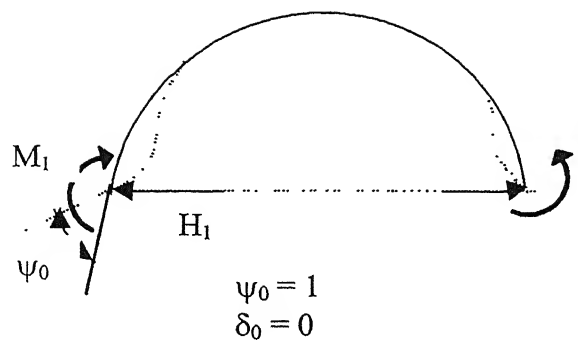


Fig. 14 Moment stiffness (M_1) and corresponding radial force (H_1) per unit length of top dome

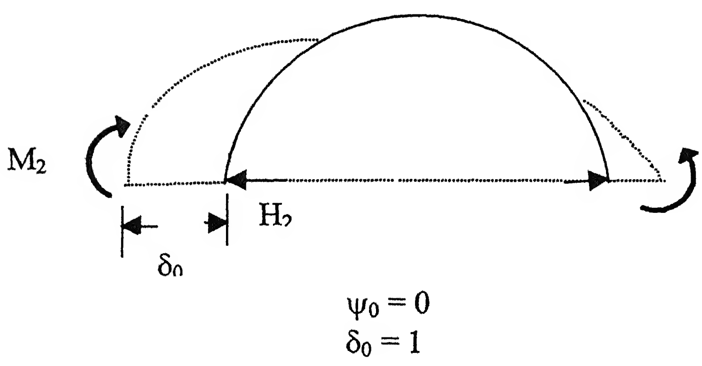


Fig. 15 Thrust stiffness (H_2) and corresponding moment (M_2) per unit length of top dome

(ii) Ring beam B_l (Fig16):

Radial thrust to cause unit outward deflection $(H_3) = E\, b_l\, d_l / R^2$ (3.92)

Moment per unit circumference to cause unit rotation $(M_3) = (E\, b_l\, d^3{}_l)/12\, R^2$ (3.93)

(iii) Cylindrical wall (Fig 17 and 18):

Moment stiffness (clockwise) $(M_4) = 2\,\mu\, Z$ (3.94)

Corresponding thrust (outward) $(H_4) = 2\,\mu^2\, Z$ (3.95)

Thrust stiffness (inward) $(H_5) = 4\,\mu^3\, Z$ (3.96)

Corresponding moment (anticlockwise) $(M_5) = 2\,\mu^2\, Z$ (3.97)

Where, $$\mu = \left(\frac{12}{D^2 t^2{}_{wave}}\right)^{\frac{1}{4}}$$

$$Z = \frac{E t^3_{wave}}{12}$$

Membrane displacement (outward-radial) of tank wall at bottom $(X_w) = \dfrac{P_2 R_1}{t_{wave} E}$ (3.98)

Clockwise slope of the wall $(\theta_w) = X_w/h$

3.2.2.1.2 Reaction due to Continuity at Joint:

Let the net rotation (clockwise) and net displacement (inward) be ψ and δ respectively. Then the changes in the slop and displacement of various members from the membrane states can be expressed as follows:

Element	Clockwise slope	Inward displacement
Top dome	$\psi - \psi_d$	$\delta - \delta_d$
Beam B_l	ψ	δ
Tank wall	$\psi - \theta_w$	δ

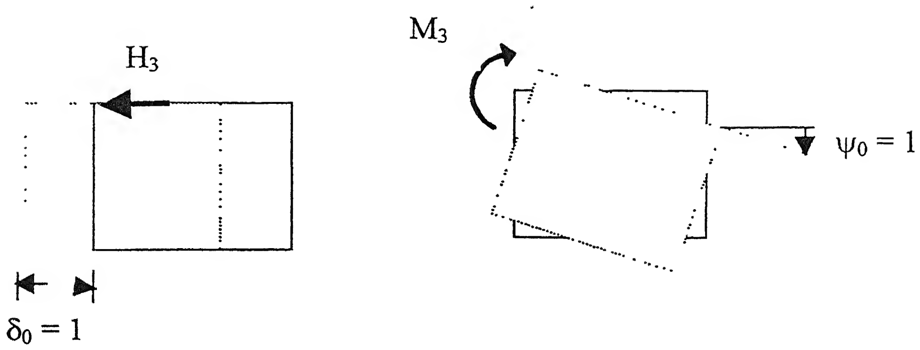


Fig 16. Radial thrust and moment of ring beam (B_1)

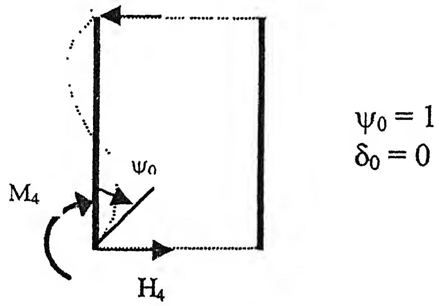


Fig. 17 Moment stiffness (M_4) and Corresponding radial force (H_4) per unit length of cylindrical wall

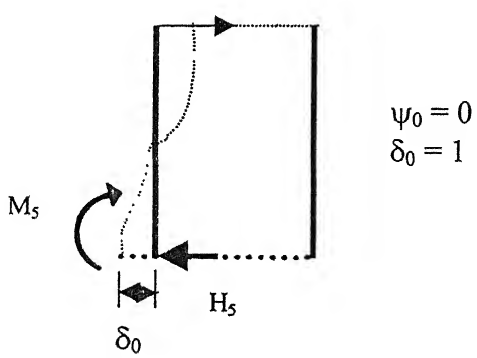


Fig. 18 Thrust stiffness (H_5) and Corresponding moment (M_5) per unit length of cylindrical wall

The reaction imposed by the joint on each member is, therefore, be equal to the product of the above change in slope/deflection and the corresponding stiffness. Since the dome imposed an outward thrust of P_1 N/m, the joint must react with an inward thrust of P_1 N/m.

Equation of consistency of deformations (joint at Ring beam B_1)

Clockwise moment:

$$M_1 (\psi - \psi_d) + H_1 (\delta - \delta_d) + M_3 \psi + M_4 (\psi - \theta_w) - H_4 \delta = 0 \qquad (3.99)$$

Inward thrust:

$$M_2 (\psi - \psi_d) + H_2 (\delta - \delta_d) + H_3 \delta - M_5 (\psi - \theta_w) + H_5 \delta + P_1 = 0 \qquad (3.100)$$

These two equations can be solved for ψ and δ, and, the reactions due to continuity can be evaluated. The reactions, F_1 to F_9 (Fig 19) as derived from these evaluations are given in the following.

Element	Moment	Thrust	Hoop tension
Dome	$F_1 = M_1 (\psi - \psi_d) + H_1 (\delta - \delta_d)$	$F_4 = M_2 (\psi - \psi_d) + H_2 (\delta - \delta_d)$	$F_7 = -(\delta E A_{ud})/(D/2)$
Ring beam (B_1)	$F_2 = M_3 \psi$	$F_5 = H_3 \delta$	$F_8 = -(\delta E A_{b1})/(D/2)$
Wall	$F_3 = M_4 (\psi - \theta_w) - H_4 \delta$	$F_6 = - M_5 (\psi - \theta_w) + H_5 \delta$	$F_9 = -(\delta E A_w)/(D/2)$

Where A_{ud}, A_{b1}, A_w are the cross section areas of the respective members for unit length.

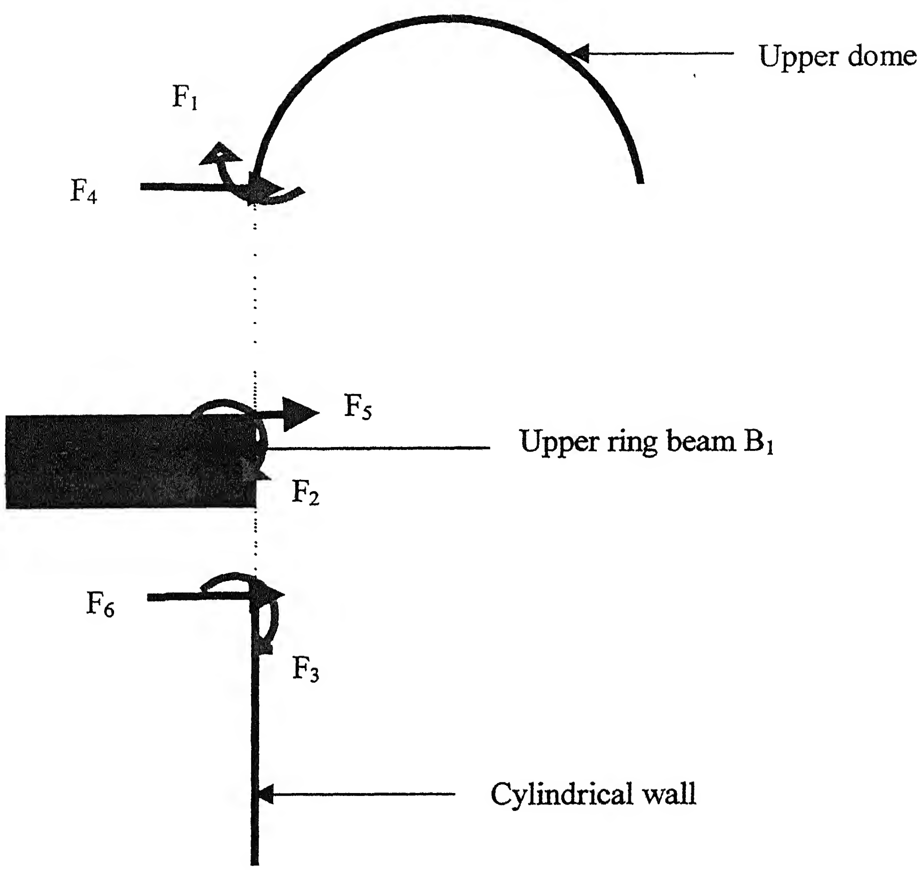


Fig. 19 Forces due to continuity at the upper ring beam B_1 joint

3.2.2.1.3 Design of member:

(i) Top dome: If the thrust F_4 is less than the meridional thrust T_1 (calculated membrane analysis) the design done using membrane analysis is safe; otherwise, redesi is to be done for thrust. Due to joint effect a hogging moment F_1 is imposed.

Bending stress per unit breadth at edge due to this moment

$$\sigma_{bd1} = \frac{6F_1}{1000 \times t^2} \qquad (3.10$$

If the direct compressive stress σ_{md} (obtained earlier in membrane analysis) is great than the bending stress σ_{bd1} (obtained from continuity analysis) the design done l membrane analysis is safe otherwise it is to be designed for bending stress.

If $\sigma_{bd1} > \sigma_{md}$,

Area of reinforcement (on one face) required to resist the tensile stress $(A_{1c}) = \dfrac{F_1}{\sigma_{st} j d'_d}$

(3.102

Where d'_d is the effective depth of the dome and j is the lever arm factor of dome.

$$\text{Spacing of the reinforcement } (S_{1c}) = \frac{1000 \times A_{1c\varphi}}{A_{1c}} \qquad (3.103$$

Where $A_{1c\phi}$ is the cross-section area of single reinforcement.

In addition to this, the dome is also to be designed for hoop tension (F_7)

Area of hoop reinforcement $(A_{2c}) = F_7 / \sigma_{st}$ (3.104)

$$\text{Spacing of the hoop reinforcement } (S_{2c}) = \frac{1000 \times A_{2c\phi}}{A_{2c}} \qquad (3.105$$

Where $A_{2c\phi}$ is the cross-section area of single reinforcement.

(ii) Ring beam B_1: If the radial force P_1 in beam calculated in membrane analysis, is greater than the inward thrust (radial force) F_5 calculated in continuity analysis, there is no need for redesigning the beam. Otherwise, it is to be redesigned for continuity hoop stress.

For $F_5 > P_1$,

$$\text{Area of steel } A_{s2} = F_5/\sigma_{st} \tag{3.106}$$

$$\text{Number of bars } N_2 = A_{s2}/A_{\phi 1}$$

$$\text{Tensile stress in equivalent section } \sigma_{tc} = F_5/\ \{b_1 \times d_1 + (m-1)\, A_{s2}\} \tag{3.107}$$

Due to joint effect a hogging moment F_2 is imposed at the edge of the beam (Generally it is too small a moment and the hoop steel, as provided, will take care of this also.).

$$\text{Area of reinforcement to resist the edge moment } F_2,\ (A_{3c}) = \frac{F_2}{\sigma_{st}\, j d'_{b1}} \tag{3.108}$$

Where d'_{b1} is the effective depth of the upper ring beam and j is the lever arm factor of the ring beam.

$$\text{Spacing of the reinforcement } (S_{3c}) = \frac{1000 \times A_{3c\varphi}}{A_{3c}} \tag{3.109}$$

3.2.2.2 Effect of continuity at the junction of Cylindrical portion, Beam (B_3) and Conical Dome:

3.2.2.2.1 Membrane Deformation and Stiffness [7]: In order to determine the effect of continuity of joints, first will be calculated the membrane deformation and stiffness at edge.

(i) Ring Beam B_3:

$$\text{Radial thrust to cause unit outward deflection } (H_6) = E\, b_3\, d_3/\, R^2 \tag{3.110}$$

$$\text{Moment per unit circumference to cause unit rotation, } (M_6) = (E\, b_3\, d_3^3)/12\, R^2 \tag{3.111}$$

(ii) Conical Dome(Fig 20 to 23):

Outward deflection at the top edge $(\delta_{ct}) = \dfrac{RP_{5t}}{Et_c}$ (3.112)

Outward deflection at bottom edge $(\delta_{cb}) = \dfrac{RP_{5b}}{Et_c}$ (3.113)

Slope at top edge $(\psi_t) = -\dfrac{\tan\phi_0\left(2P_{5t}+W_5 Sec\phi_0\right)}{Et_c}$ (3.114)

Slope at bottom edge $(\psi_b) = -\dfrac{\tan\phi_0\left(2P_{5b}+W_9 Sec\phi_0\right)}{Et_c}$ (3.115)

Stiffness at the Top Edge:

Moment stiffness $(M_7) = \dfrac{Et_c K_4 L}{\left(K_1K_4-K_2K_3\right)\tan^2\phi_0}$ (3.116)

Corresponding thrust $(H_7) = \dfrac{Et_c K_2}{\left(K_1K_4-K_2K_3\right)\tan\phi_0 Sin\phi_0}$ (3.117)

Thrust stiffness $(H_8) = \dfrac{Et_c K_1}{\left(K_1K_4-K_2K_3\right)LSin^2\phi_0}$ (3.118)

Corresponding moment $(M_8) = \dfrac{Et_c K_3}{\left(K_1K_4-K_2K_3\right)\tan\phi_0 Sin\phi_0}$ (3.119)

Stiffness at the Bottom Edge:

Moment stiffness $(M_9) = \dfrac{Et_c K_4^1 L^1}{\left(K_1^1K_4^1-K_2^1K_3^1\right)\tan^2\phi_0}$ (3.120)

Corresponding thrust $(H_9) = \dfrac{Et_c K_2^1}{\left(K_1^1K_4^1-K_2^1K_3^1\right)\tan\phi_0 Sin\phi_0}$ (3.121)

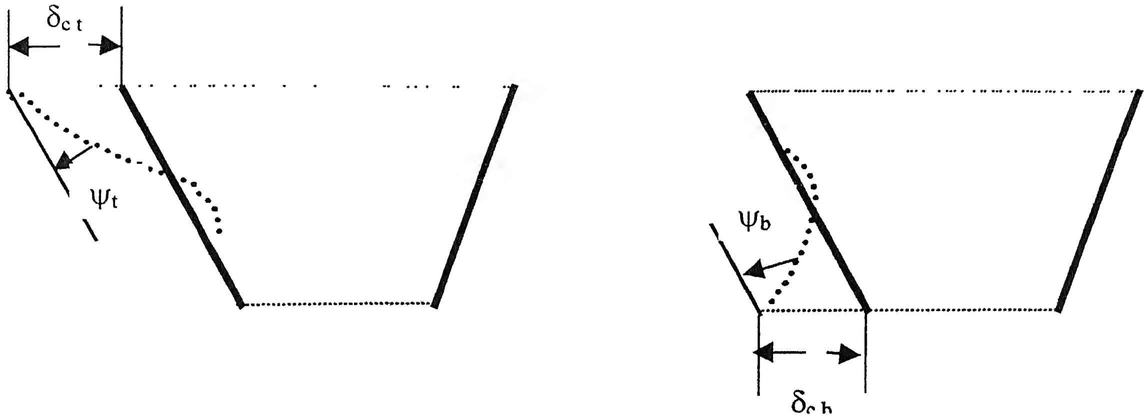


Fig. 20 slope and deflection at the top and bottom of conical dome

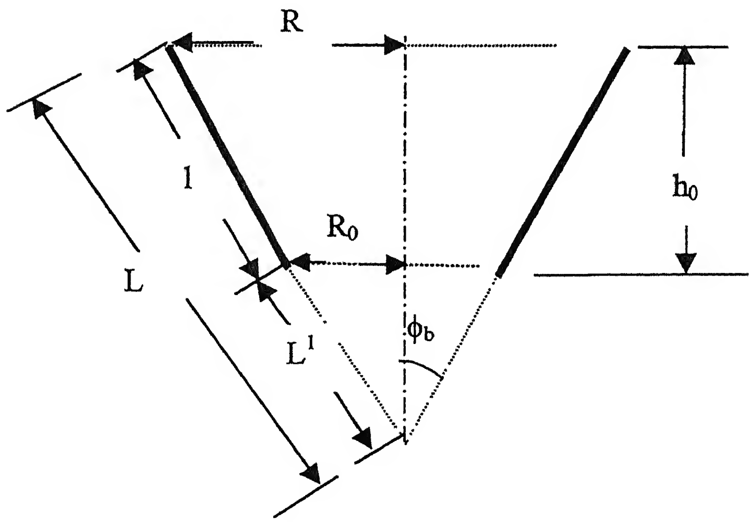


Fig. 21 Dimensions of conical dome

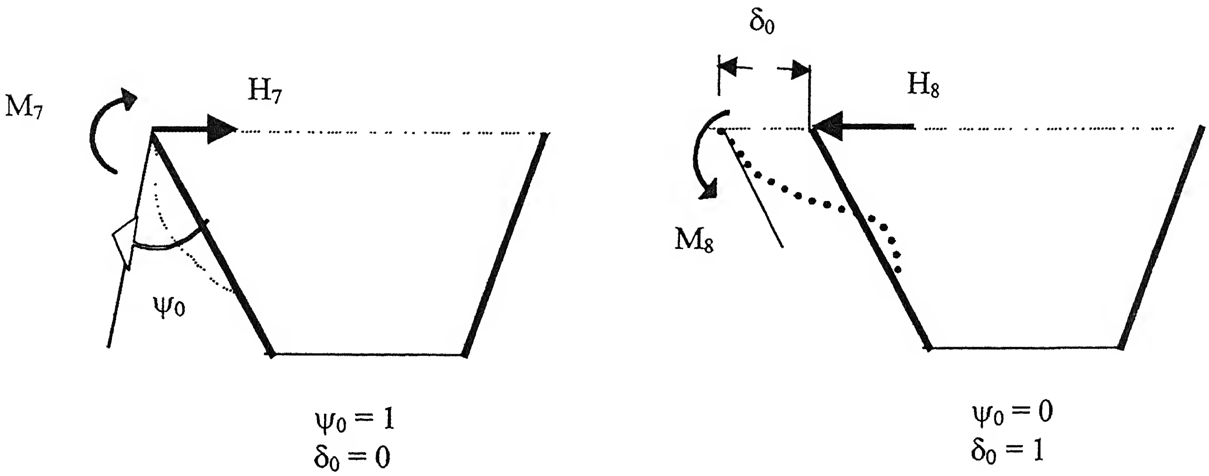


Fig 22 Stiffness at the top edge of conical dome

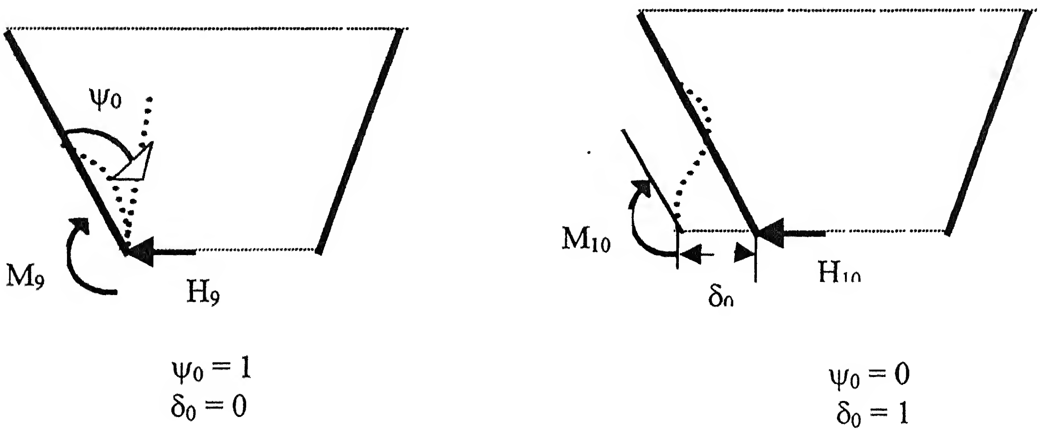


Fig 23 Stiffness at the bottom edge of conical dome

$$\text{Thrust stiffness } (H_{10}) = \frac{Et_c K_1^1}{\left(K_1^1 K_4^1 - K_2^1 K_3^1\right) L^1 Sin^2 \phi_0} \quad (3.122)$$

$$\text{Corresponding moment } (M_{10}) = \frac{Et_c K_3^1}{\left(K_1^1 K_4^1 - K_2^1 K_3^1\right) \tan\phi_0 Sin\phi_0} \quad (3.123)$$

For the given geometry of the conical shell K_1, K_2, K_3, K_4 and $K^1{}_1$, $K^1{}_2$, $K^1{}_3$, $K^1{}_4$ can be obtained from the Table 6 shown, after evaluating Z, Z^1 below. L is the slant length of the cone (complete) and L^1 is the truncated cone slant length as shown in Fig. 21.

$$Z = 2\Delta\sqrt{L}, Z^1 = 2\Delta\sqrt{L^1}$$

$$L^1 = R_0 \operatorname{cosec}\phi_0, \quad L = l + L^1, \quad l = \sqrt{h_0^2 + (R - R_0)}$$

Where, $\Delta^4 = \dfrac{12\cot^2\phi_0}{t_c^2}$

ϕ_0 is the semi apex angle of the cone.

3.2.2.2.2 Reaction due to continuity at the ring beam B_3 joint:

Let the net rotation (clockwise) and net displacement (inward) be ψ_1 and δ_1 respectively. Then the changes in the slop and displacement of various members from the membrane states can be expressed as follows

Element	Clockwise slope	Inward displacement
Wall	$\psi_1 - \theta_w$	$\delta_1 + X_w$
Beam (B_3)	ψ_1	δ_1
Conical dome	$\psi_1 + \psi_t$	$\delta_1 + \delta_t$

Table 6: Conical Shell Stiffness Constants

Z, Z^1	K_1	K_2, K_3	K_4	K^1_1	K^1_2, K^1_3	K^1_4
10	183.0	25.20	6.29	169.0	23.55	7.30
11	220.5	27.25	7.21	226.5	29.35	7.90
12	288.0	32.60	7.91	312.0	36.00	8.70
13	366.5	38.40	8.60	396.0	42.20	9.40
14	463.0	46.20	9.36	490.0	48.85	10.09
15	568.5	52.50	10.10	596.0	56.30	10.82
16	696.0	60.30	10.82	728.0	63.20	11.42
17	834.0	68.40	11.52	880.0	72.30	12.13
18	990.0	76.65	12.24	1038	81.20	12.97
19	1172	86.00	12.98	1222	89.70	13.61
20	1370	95.90	13.69	1430	100.0	14.30
21	1574	105.6	14.39	1658	111.0	15.10
22	1830	116.3	15.10	1900	121.1	15.70
23	2100	127.8	15.88	2190	134.0	16.50
24	2400	139.9	16.45	2470	145.0	17.16
25	2695	175.0	17.24	2800	156.9	17.80

The reaction imposed by the joint on each member, will therefore be equal to the product of the above change in slopes and deflection and the corresponding stiffness. In addition to these, there will be following three reactions:

(i) Balcony moment: Moment due to balcony slab at joint will be anticlockwise direction, then the reactive moment (M_{bal}) at joint will be clockwise direction. Generally the values of M_{bal} varies from 700 N-m/m to 1000 N-m/m.

(ii) Water pressure on beam height: $P_{bh} = \gamma_w \times h \times b_3$ (outward direction). Hence reactive thrust will be in inward direction.

(iii) Thrust from conical dome: $T_{con} = W_5 \tan\phi_0$ (outward direction). Hence reactive thrust will be in inward direction.

Equation of consistency of deformations (joint at Ring beam B_3):

Clockwise moment:

$$M_4 (\psi_1 - \theta_w) + H_4 (\delta_1 + X_w) + M_6 \psi_1 + M_7 (\psi_1 + \psi_t) + (-H_7)(\delta_1 + \delta_t) + M_{bal} = 0 \tag{3.124}$$

Inward thrust:

$$M_5 (\psi_1 - \theta_w) + H_5 (\delta_1 + X_w) + H_6 \delta_1 + (-M_8)(\psi_1 + \psi_t) + H_8 (\delta_1 + \delta_t) + P_{bh} + T_{con} = 0 \tag{3.125}$$

These two equations can be solved for ψ_1 and δ_1, and, the reactions due to continuity can be evaluated. The reactions, F_{10} to F_{18} (Fig 24) as derived from these evaluations are given in the following.

Element	Moment	Thrust	Hoop tension
Wall	$F_{10} = M_4 (\psi_1 - \theta_w) + H_4 (\delta_1 + X_w)$	$F_{13} = M_5 (\psi_1 - \theta_w) + H_5 (\delta_1 + X_w)$	$F_{16}=-(\delta_1 E A_w)/(D/2)$
Beam (B_3)	$F_{11} = M_6 \psi_1$	$F_{14} = H_6 \delta_1$	$F_{17}=-(\delta_1 E A_{b3})/(D/2)$
Conical dome	$F_{12} = M_7 (\psi_1 + \psi_t) - H_7 (\delta_1 + \delta_t)$	$F_{15} = - M_8 (\psi_1 + \psi_t) + H_8 (\delta_1 + \delta_t)$	$F_{18}=-(\delta_1 E A_{con})/(D/2)$

Where A_w, A_{b3} , A_{con} are the cross section areas of the respective members for unit length.

3.2.2.2.3 Design of members:

(i) **Cylindrical wall:** If the hoop tension F_{16} obtained from continuity analysis is less than the hoop tension P_2 obtained from membrane analysis, the design done by membrane analysis is safe. Otherwise it is to be designed for continuity analysis.

If $F_{16} \geq P_2$,

Area of steel at base of wall $(A_{s4}) = F_{16} / \sigma_{st}$

Providing area of reinforcement on both the faces, area of reinforcement on each face is $A_{s4} / 2$.

$$\text{Spacing of ring reinforcement at bottom } (S_2) = \frac{1000 \times A_{\phi 2}}{A_{s4}}$$

Since, the hoop tension varies linearly, the area of reinforcements and its spacing also vary from bottom to top.

The bending moments in the wall at both its top and bottom joints, cause tension on the outer face.

Area of steel due to bending moment $(A_{4c}) = \dfrac{F_{10}}{\sigma_{st}\, jd'}$ (3.126)

If the area of steel due to bending moment (A_{4c}) is less than the distribution reinforcement ($A_{s5}/2$) required according to membrane analysis, there is no need for providing the bending reinforcement (tensile reinforcement) in cylindrical wall.

(ii) Ring beam B_3: If the hoop tension in beam as calculated by membrane analysis, is less than that calculated by continuity analysis, then is no need for redesigning of beam. Otherwise, it is to be redesigned for continuity hoop stress.

For $F_{17} > P_3$,

Area of steel (A_{s6}) = F_{17}/σ_{st}

Number of bars (N_3) = $A_{s6} / A_{\phi 4}$

Tensile stress in equivalent section (σ_{tc}) = $F_{17}/ \{b_3 \times d_3 + (m-1) A_{s6}\}$

Due to joint effect a hogging moment F_{11} is imposed at the edge of the middle ring beam (Generally it is too small a moment, and, the hoop steel, as provided, will take care of this also.).

Area of reinforcement to resist the edge moment F_{11}, $(A_{5c}) = \dfrac{F_{11}}{\sigma_{st}\, jd'_{b2}}$ (3.127)

Where d'_{b2} is the effective depth of the middle ring beam and j is the lever arm factor of the ring beam.

Spacing of the reinforcement $(S_{5c}) = \dfrac{1000 \times A_{5c\varphi}}{A_{5c}}$ (3.128)

3.2.2.3 Effect of Continuity at the Junction of Conical Dome, Beam (B_2) and Bottom Dome:

3.2.2.3.1 Membrane deformation and stiffness [7]: In order to determine the effect of continuity of joints, first will be calculated the membrane deformation and stiffness at edge.

(i) Ring beam B_2:

Moment stiffness per radian, $$M_{11} = \frac{b_2 d_2^3 E}{3R_0^2} \quad (3.129)$$

Corresponding outward thrust per radian, $$H_{11} = \frac{b_2 d_2^2 E}{2R_0^2} \quad (3.130)$$

Thrust stiffness per unit movement, $$H_{12} = \frac{b_2 d_2 E}{R_0^2} \quad (3.131)$$

Corresponding edge moment per unit movement, $$M_{12} = \frac{b_2 d_2^2 E}{2R_0^2} \quad (3.132)$$

(ii) Bottom dome (Fig 25 and 26): The bottom dome is subjected to three types of loads:

(a) Self weight of dome (P^1) = $t_{bd} \times 1 \times \gamma_c$ N/m^2 (3.133)

(b) Weight of water contained above the crown level

$$p^1_w = \gamma_w \times (h + h_0 - h_2) \text{ N/m}^2. \quad (3.134)$$

(c) Radial water pressure acting radially from zero at the crown to maximum near edge.

The clockwise slopes and horizontal outward movement of the left edge, due to the above three forces are given by the following expressions:

Slope at edge $\psi_{bd}=\frac{2P^1 R_2 \sin\phi_1}{Et_{bd}}-\frac{\gamma_w R_2^2 \sin\phi_1}{Et_{bd}}$ (3.135)

Horizontal movement of edge =

$$\delta_{bd}=\frac{P^1 R_2^2 \sin\phi_1}{Et_{bd}}\left[\frac{1}{1+\cos\phi_1}-\cos\phi_1\right]-\frac{p_w^1 R_2^2 \sin\phi_1}{2Et_{bd}}+\frac{\gamma_w R_2^3 \sin\phi_1}{6Et_{bd}(1+\cos\phi_1)}\left[2\cos 2\phi_1+\cos\phi_2-3\right]$$

(3.136)

Moment stiffness $M_{13}=\frac{ER_2 t_{bd}}{4\alpha_2^3}\left(K_2+\frac{1}{K_2}\right)$ (3.137)

Corresponding thrust $H_{13}=\frac{Et_{bd}}{2\alpha_2^2 K_2 \sin\phi_1}$ (3.138)

Thrust stiffness $H_{14}=\frac{Et_{bd}}{\alpha_2 R_2 K_2 \sin^2\phi_1}$ (3.139)

Corresponding moment $M_{14}=\frac{Et_{bd}}{2\alpha_2^2 K_2 \sin\phi_1}$ (3.140)

Where $\alpha_2^4=3\left(\frac{R_2}{t_{bd}}\right)^2, K=1-\frac{1}{2\alpha_2}\cot\phi_1$

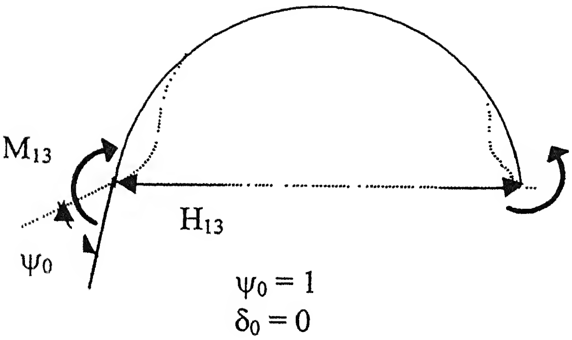


Fig. 25 Moment stiffness (M_{13}) and corresponding radial force (H_{13}) per unit length of bottom dome

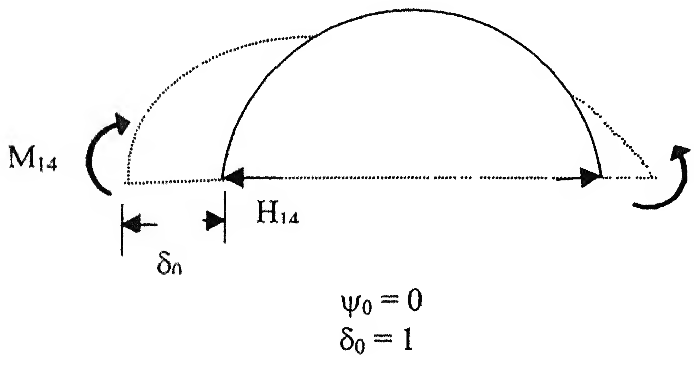


Fig. 26 Thrust stiffness (H_{14}) and corresponding moment (M_{14}) per unit length of bottom dome

3.2.2.3.2 Reaction due to Continuity at the Ring Beam B_2 Joint:

Let the net rotation (clockwise) and net displacement (inward) be ψ_2 and δ_2 respectively. Then the changes in the slop and displacement of various members from the membrane states can then be expressed as follows:

Element	Clockwise slope	Inward displacement
Conical dome	$\Psi_2 + \Psi_b$	$\delta_2 + \delta_{cb}$
Beam B_2	Ψ_2	δ_2
Bottom dome	$\Psi_2 + \Psi_{bd}$	$\delta_2 - \delta_{bd}$

The reactions imposed by the joint on each member, will therefore be equal to the product of the above change in slopes and deflection and the corresponding stiffness. In addition to this, the beam B_2 is subjected to a net inward thrust of P_6 N/m. Hence the joint reaction will be an outward force of P_6 N/m.

Equation of consistency of deformations (joint at Ring beam B_2):

Clockwise moment:

$$M_9 (\Psi_2 + \Psi_b) + H_9 (\delta_2 + \delta_{cb}) + M_{11} \Psi_2 - H_{11} \delta_2 + M_{13} (\Psi_2 + \Psi_{bd}) + H_{13} (\delta_2 - \delta_{bd}) = 0 \tag{3.141}$$

Inward thrust:

$$M_{10} (\Psi_2 + \Psi_b) + H_{10} (\delta_2 + \delta_{cb}) - M_{12} \Psi_2 + H_{12} \delta_2 + M_{14} (\Psi_2 + \Psi_{bd}) + H_{14} (\delta_2 - \delta_{bd}) - P_6 = 0 \tag{3.142}$$

These two equations can be solved for ψ_2 and δ_2, and, thus the reactions due to continuity can be evaluated. The reactions, F_{19} to F_{27} (Fig 27) as derived from these evaluations are given in the following..

Element	Moment	Thrust	Hoop tension
Conical dome	$F_{19} = M_9 (\Psi_2 + \Psi_b) + H_9 (\delta_2 + \delta_{cb})$	$F_{22} = M_{10} (\Psi_2 + \Psi_b) + H_{10} (\delta_2 + \delta_{cb})$	$F_{25}=(\delta_2 E A_{con})/(D_0/2)$
Ring beam B_2	$F_{20} = M_{11} \Psi_2 - H_{11} \delta_2$	$F_{23} = - M_{12} \Psi_2 + H_{12} \delta_2$	$F_{26} = (\delta_2 E A_{b2})/(D_0/2)$
Bottom dome	$F_{21} = M_{13} (\Psi_2 + \Psi_{bd}) + H_{13} (\delta_2 - \delta_{bd})$	$F_{24} = M_{14} (\Psi_2 + \Psi_{bd}) + H_{14} (\delta_2 - \delta_{bd})$	$F_{27}=-(\delta_2 E A_{bd})/(D_0/2)$

Where A_{bd} is the cross sectional area of bottom dome.

3.2.2.3.3 Design of members

(i) Conical dome: If the hoop tension as calculated in membrane analysis is greater than then obtained from continuity analysis, the dome is not to be redesigned; otherwise, it is to be redesigned based on the hoop tension calculated in continuity analysis.

For $F_{25} > T_{4\,max}$,

$$\text{Area of hoop reinforcement } (A_{s7}) = F_{25}/\sigma_{st} \qquad (3.143)$$

$$\text{Area of hoop reinforcement on each face} = A_{s7}/2$$

$$\text{Spacing of bars on each face } (S_4) = (1000 \times A_{\phi 5}) \times 2/A_{s7} \qquad (3.144)$$

$$\text{Maximum tensile stress in composite section } (\sigma_{ct}) = \frac{F_{25}}{(b_c \times d_c) + (m-1)A_{s7}} \qquad (3.145)$$

σ_{ct} should be less than the permissible value.

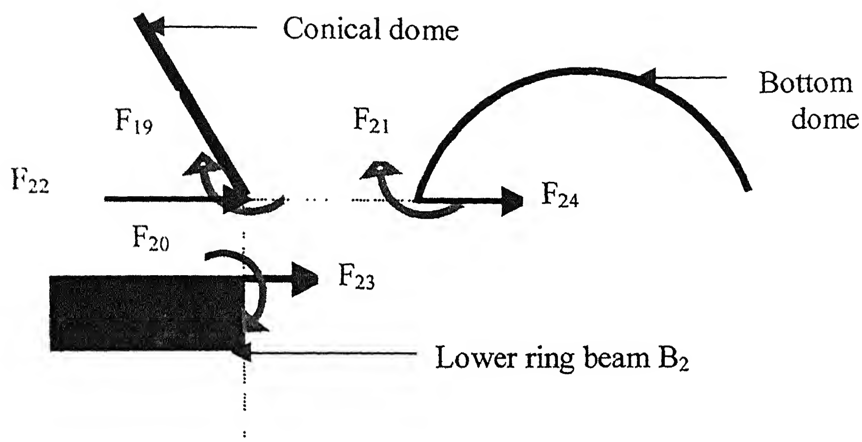


Fig. 27 Forces due to continuity at the upper ring beam B_2 joint

If σ_{ct} is greater than the permissible value, the thickness of conical dome is to be increased.

Bending moment at the bottom edge = F_{19}

Area of reinforcement due to bending moment $(A_{6c}) = \dfrac{F_{19}}{\sigma_{st} jd'}$ (3.146)

Spacing of reinforcement $(S_{6c}) = \dfrac{1000 \times A_{3c\phi}}{A_{4c}}$ (3.147)

This reinforcement is provided on water face @ S_{3c} mm c/c. Two third reinforcement may be curtailed at a distance 42ϕ from the joint.

(ii) Bottom dome:

The bottom dome is subjected to the following forces:

Hoop tension $= F_{27}$

Meridional thrust $= F_{24}$

Bending moment $= F_{21}$

Tensile stress on bottom dome $(T_{ts}) = \dfrac{F_{21} \times 6}{t_{bd1}^2} - \dfrac{F_{24}}{t_{bd1}}$ (3.148)

The thickness obtained from above expression (3.148) is generally greater than the previous thickness (obtained from membrane analysis). Hence the thickness is to be increased from t_{bd} to t_{bd1} near the junction, gradually over a length of $0.76\sqrt{R_o t_{bd1}}$.

Area of reinforcement due to bending moment $(A_{7c}) = \dfrac{F_{21}}{\sigma_{st} j t_{bd1}}$ (3.148)

Spacing of reinforcement $(S_{7c}) = \dfrac{1000 \times A_{4c\phi}}{A_{5c}}$ (3.150)

These reinforcement are provided near the water face meridionally to resist the compressive bending stresses developed due to continuity bending moment.

(iii)Ring beam B_2 : The ring beam is subjected to a radial force of F_{23} and moment of F_{20}, both acting at the top edge. Hence the beam is subjected to a net moment along its circumference. This gives rise to a moment at every section. These reactions (radial force F_{23} and moment F_{20}) should be considered along with the bending and shear force obtained from membrane analysis. Generally the hoop tension caused due to reactions (radial force and moment) is negligibly small.

$$\text{Net bending moment } (M_{t,h,s}) = \left(\pm F20 \mp F23 \times \frac{b2}{2}\right) \times \frac{D0}{2} \tag{3.151}$$

Maximum hogging moment at support from membrane analysis $= M_{max}$

$$\text{Hence total net moment } (M_{tn}) = M_{max} \pm M_{t,h,s} \tag{3.152}$$

$$\text{Area of reinforcement at support } (A_{s\,12}) = M_{nt}/(\sigma_{st}\, j\, d_{2\,f}) \tag{3.153}$$

$$\text{Numbers of bars at support } (N_6) = A_{s\,12}/A_{\phi 7} \tag{3.154}$$

3.3 Analysis and Design of Staging:

The typical structural arrangement of staging is illustrated in Fig. 28. Staging consists of R. C. columns of circular cross-section placed axi-symmetrically on the circumference of a circle, braced together by horizontal bracing members located at some spacing in the vertical direction. The columns are subjected to the vertical gravity loads and lateral forces due to wind or earthquakes.

3.3.1 Analysis of staging

3.3.1.1 Vertical Load Analysis (Fig 28):

Let the tank be supported on n number of columns, symmetrically placed on a circular circumference. The height of staging above ground level is y m. The staging can be dividing in n_l panels.

Height of each panels (x) =y/n_l.

The columns be connected to raft foundation by means of ring beam, the top of which is provided at x_l m below the ground level so that the actual height of bottom panel is $(x + x_l)$ m.

Vertical load on column (W_{18}) = Total load on bottom ring beam B_2 per m $\times \pi \times D_0$

$$= W_{17} \times \pi \times D_0 \tag{3.155}$$

Load per columns (W_{19}) = W_{18}/n

$$\text{Self weight of column per m } (W_{20}) = \pi/4 \times d^2_c \times 1 \times \gamma_c \tag{3.156}$$

$$\text{Self weight of each brace } (W_{21}) = b_r \times d_r \times l_e \times \gamma_c \tag{3.157}$$

Where l_e clear length of each brace.

$$\text{Length of each brace } (L_e) = \frac{R\,Sin\frac{2\pi}{n}}{Cos\frac{2\pi}{n}} \tag{3.158}$$

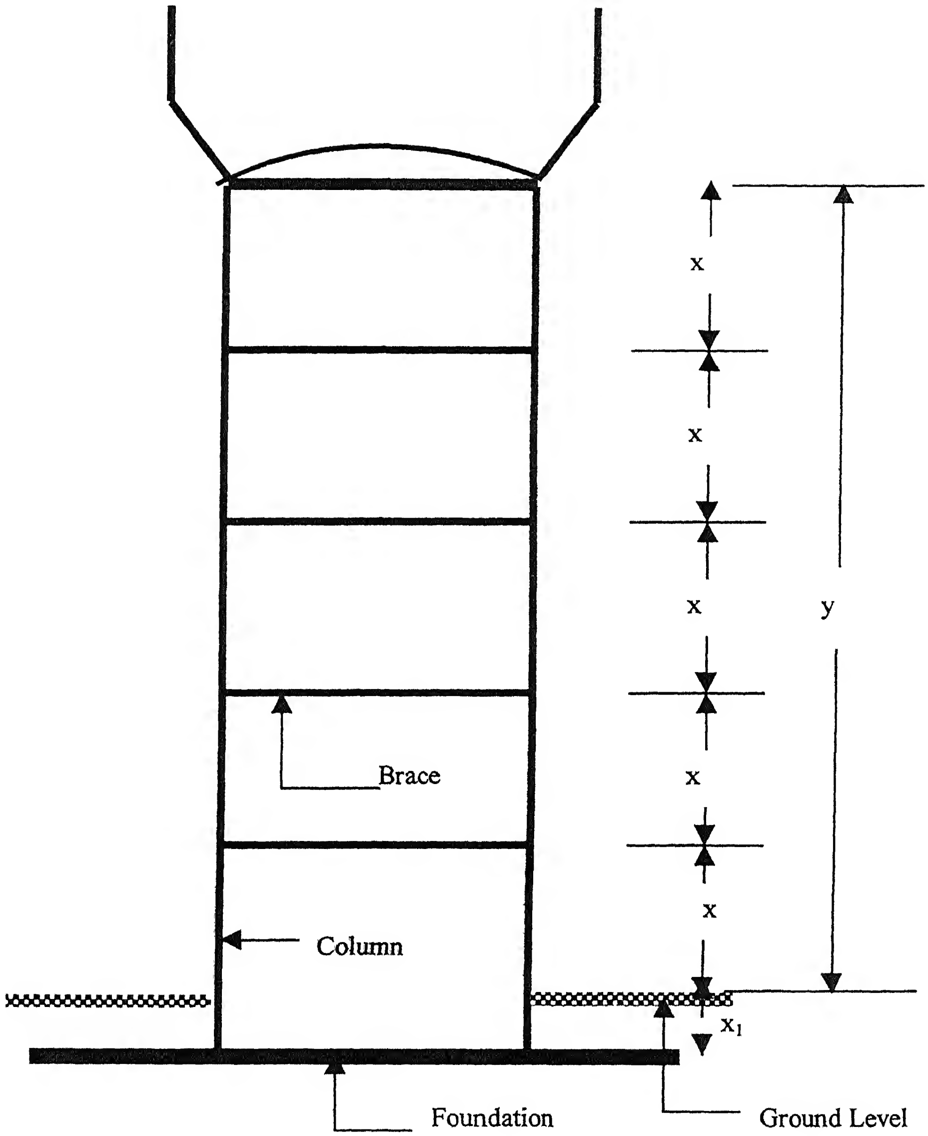


Fig 28 Structural Arrangement of the staging.

Clear length of each brace (l_e) $=L_e - d_c$

The total weight of columns just above the each brace can be expressed in the following terms:

For $(n_l - 1)$ panels, $W_i^1 = W_{19} + ixW_{20} + (i-1)W_{21}$ (3.159)

Where i =1, 2……..(n_l- 1)

For last panel, i.e. $i=n_l$

$$W_i^1 = W_{19} + (ix + x_1)W_{20} + (i-1)W_{21} \tag{3.160}$$

3.3.1.2. Wind load analysis:

Since the tank base, foundation and the braces are very stiff compared to the columns it is considered that the tower deflects, maintaining the column axes almost vertical at their top, bottom and at their junctions with braces developing the points of inflection at mid height of each panel [7]. The deflected pattern of the tower is shown in Fig. 29.

Intensity of wind pressure [17] $p_y = 0.6\, V^2_y$ N/m^2

Where V_y = design wind velocity in m/s at height y.

The wind force on various elements of the container can be expressed as follows (Fig 30).

Tank components	Projected area	Wind load
Dome	$A_1 = \frac{4h_1}{3}\sqrt{\frac{D^2}{4} + \frac{2h_1^2}{3}}$	$W_{wd} = A_1 \times \beta \times p_y$
Cylindrical wall	$A_2 = h \times D$	$W_{ww} = A_2 \times \beta \times p_y$
Conical dome	$A_3 = (H_0 - h) \times (D + D_0)/2$	$W_{wcd} = A_3 \times \beta \times p_y$
Lower ring beam	$A_4 = D_0 \times d_2$	$W_{wlb} = A_4 \times \beta \times p_y$

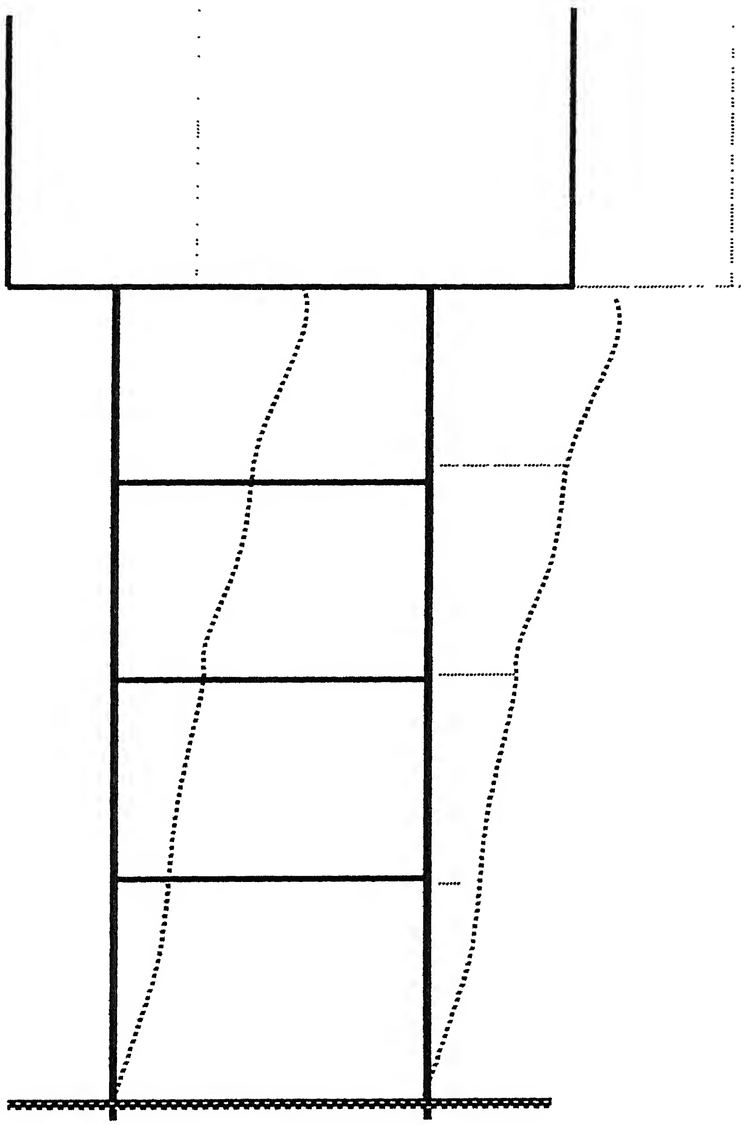


Fig. 29 Deflected pattern of the water tank

β = Shape factor on the tank

$$\text{Total wind load } (W_{wt}) = W_{wd} + W_{ww} + W_{wcd} + W_{wlb} \tag{3.161}$$

I may be assumed that the resultant wind force act at mid height of the cylindrical wall.

Height of the resultant wind force above the base of the bottom ring beam,

$$h_3 = (h/2 + h_0 + d_3)$$

$$\text{Wind load on each panel } (W_{pan}) = x \times d_c \times n \times \beta \times p_y + d_r \times D_0 \times \beta \times p_y \tag{3.162}$$

$$\text{Wind load at the top end of panel } (W_{tpan}) = (x \times d_c \times n \times \beta \times p_y)/2 \tag{3.163}$$

Analysis of column:

Shear force and moment of each panel can be expressed in the following form.

$$\text{Shear force } (Q^w_i) = W_{wt} + W_{tpan} + (i-1)\, W_{pan} \tag{3.164}$$

Where $i = 1, 2, 3 \ldots n_l$

Moment $(M^w_i) = W_{wt} \times \{h_3 + x/2 + (i-1)\,x\} + W_{tpan} \times \{(i-1)\,x + x/2\} + (i-1)\, W_{pan} \times$

$\{k\,(k-1)/2 + k/2\}x$. for $i < n_l$, where $k = i - 1$ (3.165)

For $i = n_l$,

$M^w_{nl} = W_{wt} \times \{h_3 + (x_l + x)/2 + (i-1)\,x\} + W_{tpan} \times \{(i-1)\,x + (x_l + x)/2\} + (i-1)\, W_{pan}$

$\times \{k\,(k-1)/2 + k/2\}x$. , where $k = i - 1$ (3.166)

$$\text{The bending stress on the furthest leeward side } (f_{bi}) = \frac{4M_i}{naD_0} \tag{3.167}$$

$$\text{The axial thrust on the furthest leeward side } (V_i) = \frac{4M_i}{nD_0} \tag{3.168}$$

Where $i = 1, 2, 3, \ldots n_l$

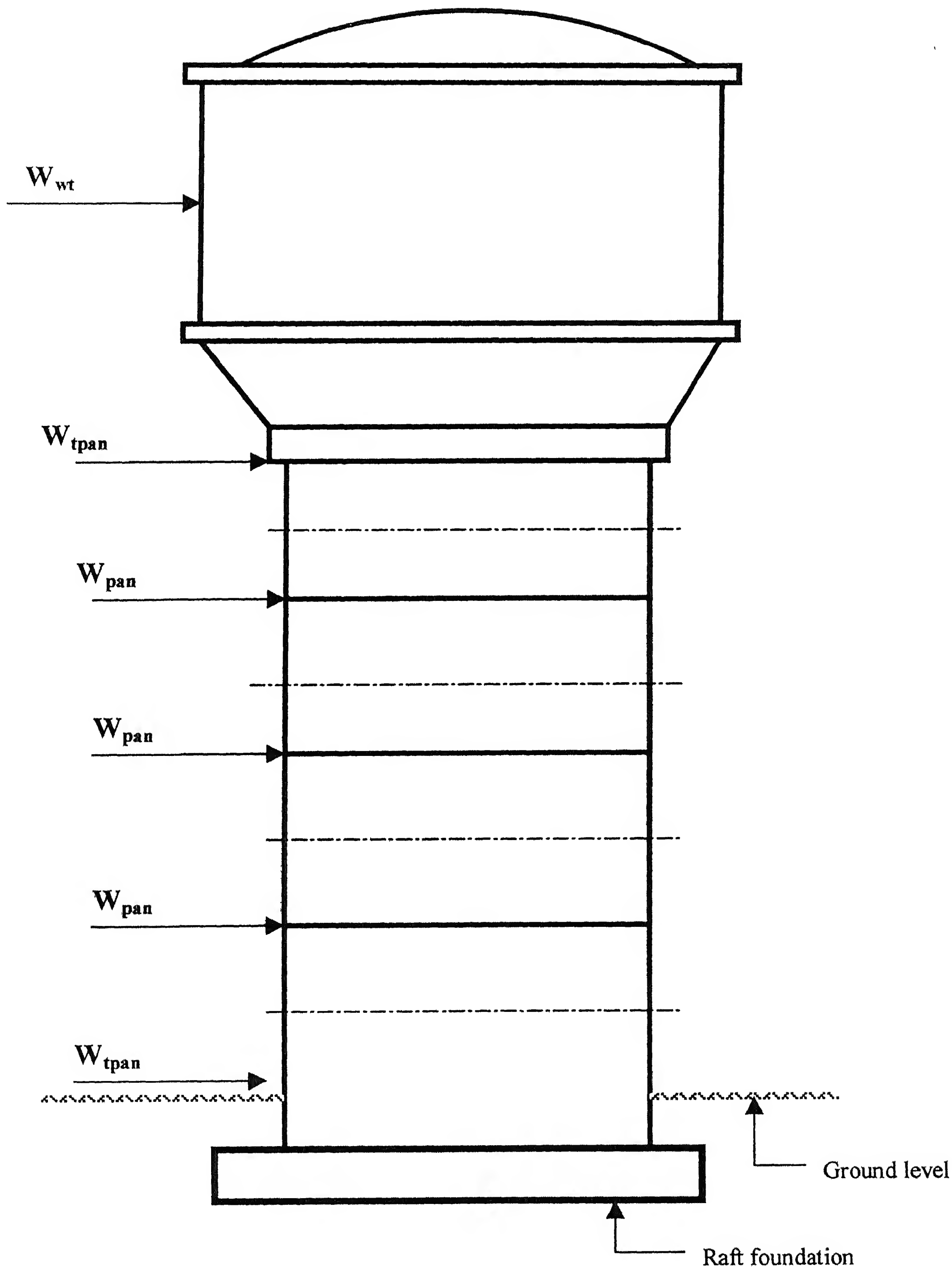


Fig. 30 Wind forces on the water

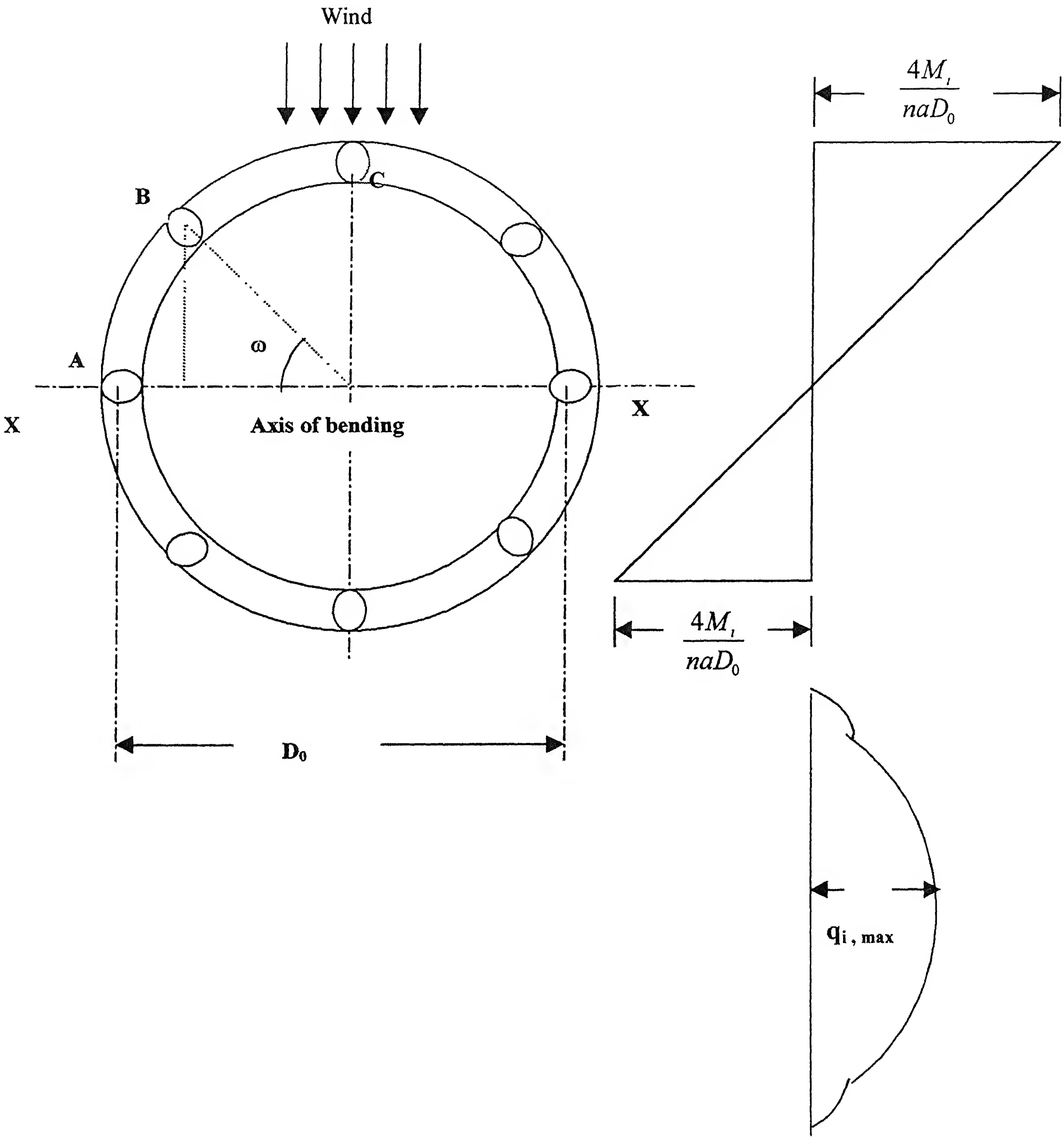


Fig. 31 Bending stress and Shear stress in column

Shear stress at any point-making angle ω with bending axis

$$q_i = \frac{2Q_i Cos^2\omega}{na} \tag{3.169}$$

Shear stress will be maximum at ω = 0, and maximum value is given by:

$$q_{i,\max} = \frac{2Q_i}{na} \tag{3.170}$$

$$\text{Maximum shear force } (S_{i,\max}) = \frac{2Q_i}{n} \tag{3.171}$$

$$\text{Maximum bending moment } (M_{i,\max}) = \frac{S_{i,\max} \times x}{2} \tag{3.172}$$

Analysis of Braces: The shear force acting on either side and the moment set up at the joint are shown in Fig. 32.

The maximum bending moment occurs in lower panels.

$$\text{Maximum bending moment: } (m_{n1}) = \frac{Q_{n1}^{w} \times (x + x_1) + Q_{(n1-1)}^{w} \times x}{nSin\frac{2\pi}{n}} Cos^2\theta Sin\left(\theta + \frac{\pi}{n}\right)$$

(3.173)

Where θ is determined graphically from the following expression:

$$\tan\left(\theta + \frac{\pi}{n}\right) = \frac{1}{2} Cot\theta \tag{3.174}$$

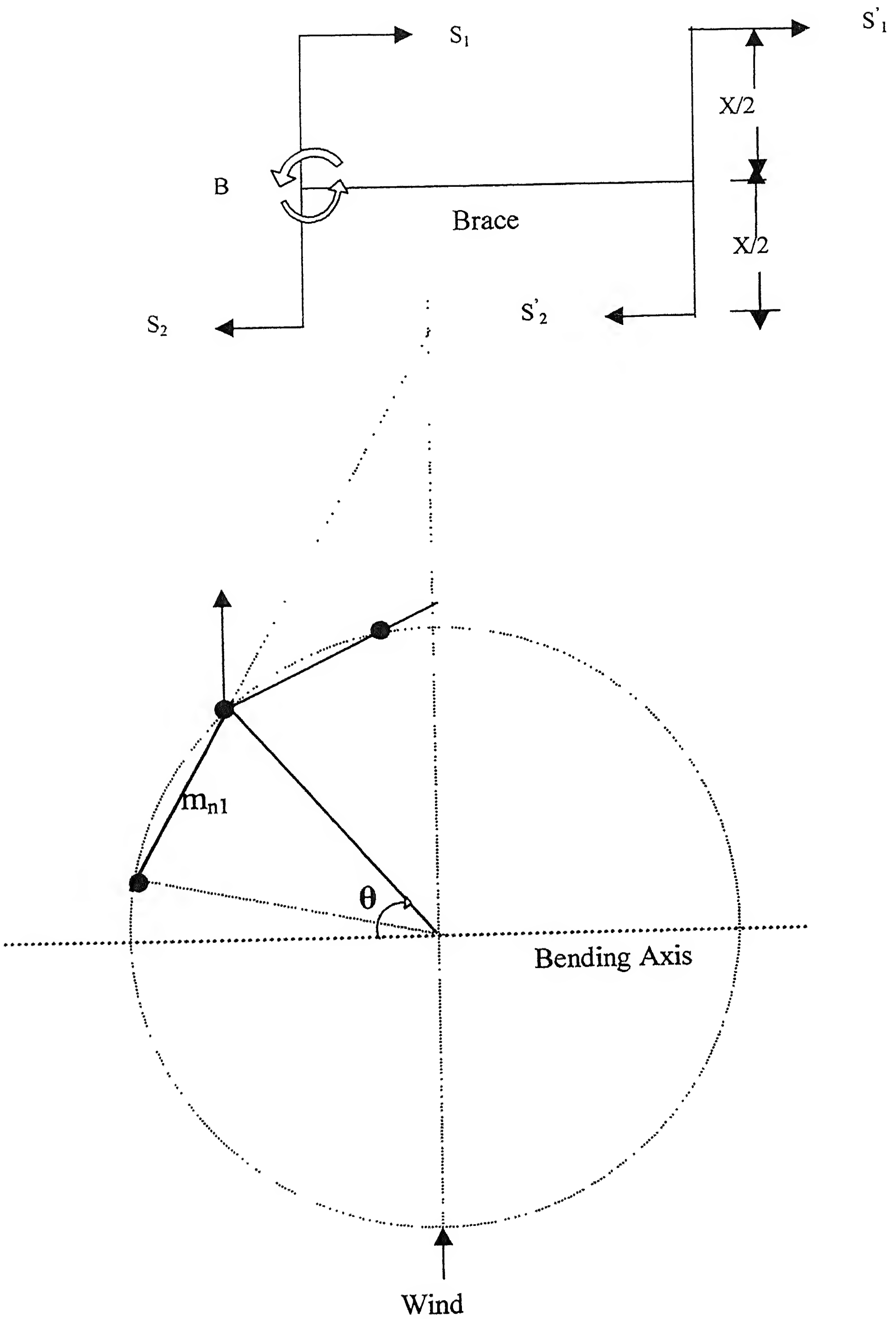


Fig.32 Shear and moment in the bracing beam

The maximum shear force in lower brace

$$S_{b,\max} = \frac{Q_{n1}^{w} \times (x+x_1) + Q_{(n1-1)}^{w} \times x}{L_e n Sin\frac{2\pi}{n}} (2Cos^2\frac{\pi}{n} Sin\frac{2\pi}{n}) \qquad (3.175)$$

In addition to bending moment and shear force, the brace is also subjected to twisting moment, the value of which may be taken equal to 5 percent of maximum bending moment [7]. The value of maximum bending moment and shear force both should not occur simultaneously.

Design twisting moment (M^t_b) = 0.05 m_{n1}

Thus brace will be subjected to a critical combination of maximum shear force and a twisting moment when wind blows a parallel to it. The brace is reinforced equally at top and bottom since the sign of moment depends upon the direction of wind.

3.3.1.3 Earthquake Load Aanalysis:

The staging tower should be designed for the forces and moments due to earthquake loading also. The design forces can be obtained by any one of the two methods specified in IS: 1893 (criteria for earthquake resistant design of structures)[15], (i) Seismic coefficient method, (ii) Response spectrum method. The earthquake factor is calculated and the earthquake force is obtained which is coming as shear in the columns. The design values of horizontal seismic coefficient, α_h in the seismic coefficient and response spectrum methods shall be computed as given by the following expressions:

i) In Seismic Coefficient Method the design value of horizontal seismic coefficient α_h shall be computed as given by following expression:

$$\alpha_h = \beta \, I \, \alpha_0 \qquad (3.176)$$

Where, β = coefficient depending upon the soil- foundation system as given in Table 3 of

IS: 1893 [15]

I = factor depending upon the importance of the structure as given in Table 4 of

IS: 1893 [15]

α_0 = basic horizontal seismic coefficient as given in table 2.of IS: 1893 [15]

ii) In Response Spectrum Method the response acceleration coefficient is first obtained for the natural period and damping of the structure, and, the design value of horizontal seismic coefficient is computed using the following expression:

$$\alpha_h = \beta I F_0 \frac{S_a}{g} \tag{3.177}$$

Where β = Coefficient depending upon the soil-foundation system

I = coefficient depending upon the importance of structure

F_0 = seismic zone factor for average acceleration spectra as given in Table 2 of IS: 1893 [15].

S_a/g = average acceleration coefficient depending on the damping and appropriate natural period.

According to IS: 1893, the damping in elevated concrete tank may be considered 5 percent of the critical.

$$\text{Natural period of vibration } (T) = 2\pi\sqrt{\frac{\Delta}{g}} \tag{3.178}$$

Where, Δ = the static horizontal deflection at the top of the tank under a static horizontal force equal to weight of the tank (W) acting at center of gravity of tank.

g = acceleration due to gravity.

$\Delta = m_s g/K_s$,

Where, K_s = stiffness of staging

m_s = mass of tank.

In empty condition, m_s = mass of tank shell +one third mass of staging. (In full condition weight of water is to be added to the weight under empty condition.).

The stiffness of staging may be obtained by adding axial flexibility to the flexural flexibility.

$$\frac{1}{K_s}=\frac{1}{K_{flexural}}+\frac{1}{K_{axial}} \tag{3.179}$$

$$\frac{1}{K_{axial}}=\frac{2}{nA_cE_cr_c^2}\sum_{i=1}^{n_1}H_i^2x_i^2 \tag{3.180}$$

Where H_i is the height of the point of application of resultant lateral load from point of contra flexure of the i^{th} panel.

Flexural stiffness of intermediate panel

$$K_{i,f,panel}=\frac{12E_cI_cn}{x^3}\left[\frac{\frac{E_bI_b}{L_e}}{\frac{E_bI_b}{L_e}+\frac{2E_cI_c}{x}}\right] \tag{3.181}$$

Flexural stiffness of the uppermost and the bottom most panels

$$K_{u\&b,f,panel}=\frac{12E_cI_cn}{x^3}\left[\frac{\frac{E_bI_b}{L_e}}{\frac{E_bI_b}{L_e}+\frac{E_cI_c}{x}}\right] \tag{3.182}$$

Overall stiffness of the staging due to flexural deformation in column

$$\frac{1}{K_{flexural}}=\sum\frac{1}{K_{i,f,panel}}+\sum\frac{1}{K_{u\&b,f,panel}} \tag{3.183}$$

Earthquake force (F_{lat}) $=\alpha_h\, m_s$ (3.184)

$$\text{Design axial force in i}^{\text{th}}\text{ panel column}\,(F_i) = \frac{2F_{lat}H_i}{nr_c} \tag{3.185}$$

$$\text{Design shear in brace at i}^{\text{th}}\text{ bracing level}\,(V_{bi}) = \frac{F_{lat}Y_i}{nr_c}\operatorname{cosec}\left(\frac{\pi}{n}\right) \tag{3.186}$$

$$\text{Design bending moment in bracing at i}^{\text{th}}\text{ bracing level}\,(M_{bi}) = V_{bi} \times \frac{L_e}{2} \tag{3.187}$$

$$\text{Design shear in intermediate panel column}\,(V_c) = \frac{2F_{lat}}{n}\cos^2\left(\frac{\pi}{n}\right) \tag{3.188}$$

$$\text{Design bending moment in intermediate panel column}\,(M_c) = V_c \times \frac{x}{2} \tag{3.189}$$

$$\text{Design shear in end panel column }(V_{ce}) = \frac{F_{lat}}{nx}\left\{4\bar{y}\cos^2\left(\frac{\pi}{n}\right) + \frac{Y_i E_c I_c L_e}{3E_b I_b x}\right\} \tag{3.190}$$

$$\text{Design bending moment in end panel column }(M_{ce}) = \frac{F_{lat}}{n}\left\{2\bar{y}\cos^2\left(\frac{\pi}{n}\right) + \frac{Y_i E_c I_c L_e}{3E_b I_b x}\right\} \tag{3.191}$$

Where $\bar{y}$ is the distance of point of inflection of the end point panel from the braced end.

3.3.2 Design of staging

Load combinations for design of staging:

i) Dead load +live load

ii) Dead load +live load + wind load

iii) Dead load + live load + earthquake load

According to the above load combinations, first, the axial loads, shear forces and moments in the different components of staging, are to be determined following the

methods discussed in section (3.3.1). After calculating all the forces and moment according to above load combinations, the next step is to select the critical force combination for each element for its design.

3.2.2.1 Design of column

Maximum axial load on column = W_{col}

Maximum moment in column = M_{col}

Equivalent Area of columns (A_{ec}) = $\pi/4 \times d^2_c + (m-1) A_{sc}$ (3.192)

Equivalent moment of inertia (I_e) = $\pi/64 \times d^4_c + (m\text{-}1) A_{sc} \times (d_c - 2 d^1_c)^2 /8$ (3.193)

Where A_{sc} is the area of reinforcement provide in column and d^1_c is the effective cover.

Direct stress in column (σ^1_{cc}) = W_{col}/ A_{ec} (3.194)

Bending stress in column (σ^1_{cbc}) = ($M_{col}/ I_e) \times d_c/2$ (3.195)

For the safety of column at bending axis, the following condition must be satisfied, based on uncracked section [16].

$$\frac{\sigma'_{cc}}{1.33\sigma_{cc}} + \frac{\sigma'_{cbc}}{1.33\sigma_{cbc}} \leq 1 \qquad (3.196)$$

If the above equation is not get satisfied then diameter of column is to be increased or the column is to be designed for cracked section.

Cracked design of column (From fig 38):

Axial load capacity $W_{col} = C_c + C_s - T_r$ (3.197)

Where C_c = compressive force in concrete

$$= \frac{d_c^2 c}{2(1+\cos\beta_1)}\left[\frac{\sin^3\beta_1}{3} + \frac{\pi-\beta_1}{2}\cos\beta_1 - \frac{\cos\beta_1}{4}\sin 2\beta_1\right]$$

C_s = compressive force in compression steel *i*

$$= \sum (1.5m - 1) A_{sci} c_{sci}$$

A_{sci} = area of compression reinforcement i

c_{sci} = stress in concrete at the level of reinforcement i

T_r = tensile force in tension reinforcement i

$$= \sum A_{sti} c_{sti}$$

A_{sti} = area of tension reinforcement i

c_{sti} = stress in tension reinforcement i

Moment capacity of the section (taking the moment force about the center of the section):

$$M_{col} = \frac{d_c^3 c}{4(1+\cos\beta_1)} \left[\frac{\pi - \beta_1}{8} + \frac{\sin 4\beta_1}{32} + \frac{\cos\beta_1}{3} \sin^3\beta_1 \right] + \sum (1.5m-1) A_{sci} c_{sci} y_{sci} + \sum A_{sti} c_{sti} y_{sti} \qquad (3.198)$$

where y_{sci} = distance of compresion reinforcement i from the centre of the section

y_{sti} = distance of tension reinforcement i from the centre of section

$$\beta_1 = \cos^{-}\left(\frac{2kd_c'}{d_c} - 1 \right)$$

These two euations (3.197 and 3.198) can be solved (by using trial method) for stress in concrete and reinforcement. If the stresses are smaller than the permissible stress the design done is safe otherwise reinforcement or diameter of column is to be increased.

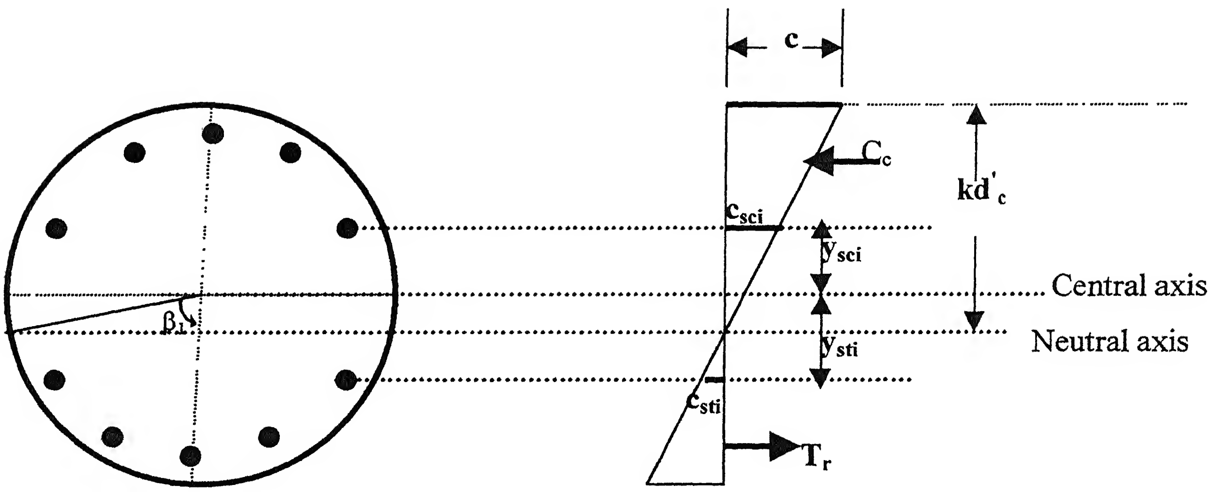


Fig 33 stress and force diagram in column

3.2.2.2 Design of braces

Maximum shear force on column = V_{bra}

Maximum moment in column = M_{bra}

Twisting moment (M^t_{bra}) = 0.05 M_{bra}

3.2.2.2.1 Main reinforcement:

The section of the brace is $b_r \times d_{rf}$ (effective) and has reinforcement equal to pb_rd_{rf} at top and bottom each at an effective cover of d'_r, then for the section to be balanced, the neutral axis will be at kd_{rf} from the compression face as shown in Fig. 34.

Equating moment of the effective area about the neutral axis to zero ,

$$0.5\, b_r (kd_{rf})^2 + (m-1)\, pb_rd_{rf} [kd_{rf} - d'_r] = m\, pb_rd_{rf} [d_{rf} - kdrf] \tag{3.199}$$

From the above equation (3.197), value of p is to be found. After knowing the value of p the depth of brace is checked, for its adequacy.

Since the brace is subjected to both the bending moment as well as twisting moment, the equivalent moment is [16]

$$M_{e1} = M_{bra} + M^t_{bra}\left(\frac{1+\frac{d_4}{b_r}}{1.7}\right) \tag{3.200}$$

In order to find the depth of the section, equating the moment of resisting of the section to the external moment (From fig 34.)

$$b_r \times kd_{rf} \times \frac{c}{2}\left[d_{rf} - \frac{kd_{rf}}{3}\right] + (1.5m-1)\times pb_rd_{rf} \times \frac{c(kd_{rf} - d'_r)}{kd_{rf}} \times (d_{rf} - d'_r) = M_{e1} \tag{3.201}$$

From the above equation (3.199), the effective depth of brace is to be found and compared with the original value. If the effective depth thus found is greater than its original value, the section is increased to a suitable value. After that the compression and tension reinforcements are found.

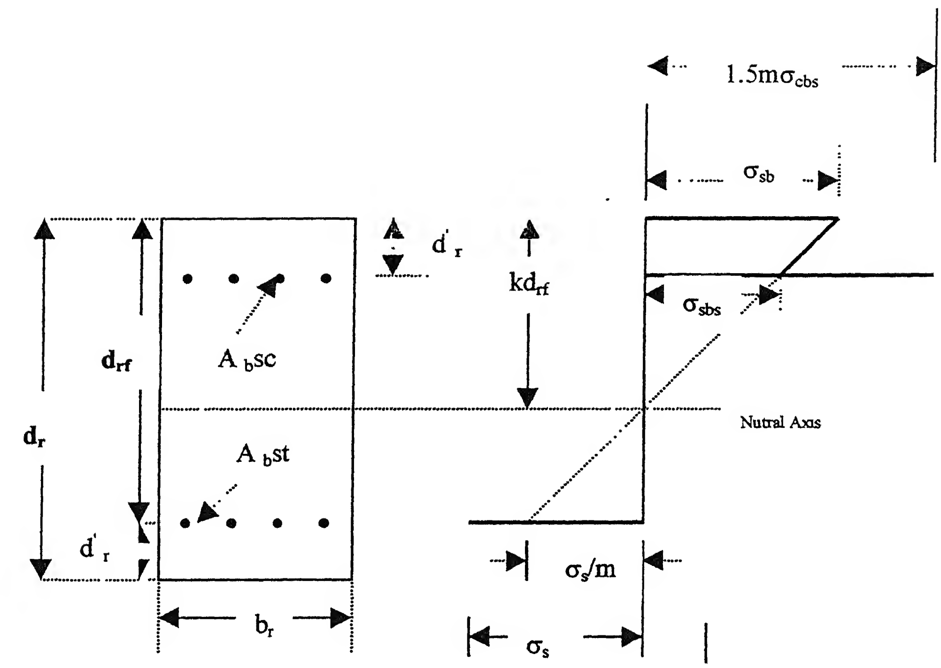


Fig. 34 Section and Stresses for staging braces.

Area of compression reinforcement (A_{bsc}) = pb_rd_{rf}

Area of tension reinforcement (A_{bst}) = pb_rd_{rf}

Number of bars of compression or tensile reinforcement (N_b) = $\frac{A_{bsc} or A_{bst}}{A_{\varphi 9}}$

3.2.2.2.2 Transverse Reinforcement

Equivalent shear force (V_e) = $V_{bra} + 1.6\, M^t_{bra}/\, b_r$ (3.202)

Shear stress ($\tau_{v\,e}$) = $V_e/b_r\, d_r$ (3.203)

$\tau_{v\,e}$ should always be less than $\tau_{c\,max}$ otherwise the section is to be redesigned [16].

If $\tau_{v\,e} > \tau_c$, shear reinforcement is necessary, otherwise minimum shear reinforcements are provided [16].

Cross-section area of the stirrups [16]

$$A_{bsv} = \frac{M^t_{bra} \times S_v}{b^1 d^1 \sigma_{sv}} + \frac{V_{bra} \times S_v}{2.5 d^1 \sigma_{sv}} \qquad (3.204)$$

Where b^1 = center- to- center distance between corner bars in the direction of the width.

d^1 = center- to- center distance between corner bars in the direction of depth.

But the total reinforcement shall not be less than $(\tau_{v\,e} - \tau_c)\, b_r\, S_v/\sigma_{sv}$

The spacing of the stirrups shall not exceed the least of x'_1, $(x'_1+y_1)/4$, and 300 mm, where x'_1 and y_1 are the respectively the shorter and long dimension of the stirrup.

Side face reinforcement: where the depth of the web in a beam exceeds 450 mm, side face reinforcement shall be provided along the two faces. The total area of such reinforcement shall not be less than 0.2 percent of the web area and shall be distributed equally on two faces at spacing not exceeding 300mm or web thickness whichever is less [16].

$A_{bsd} = (0.2 \times b_b \times d_r)/100$ mm (3.205)

3.4 Foundation Analysis and Design:

Based on the information obtained from the preliminary design, the foundation is analysed with the aid of exact methodology. Subsequently, the design of foundation is performed adequately.

Vertical load from filled tank and column (W_{22}) = $n \times W^1{}_{n1}$ (3.206)

Weight of water (W_{23}) = $W_7 + W_{11}$ (3.207)

Vertical load of empty tank and column (W_{24}) = $W_{22} - W_{23}$ (3.208)

Self weight of foundation = W_{25}

Total load (W_{26}) = $W_{22} + W_{25}$ (3.209)

Maximum moment at the base of the slab = M_{bs}

Bearing pressure of soil = q

Area of foundation required (A_f) = W_{26} / q (3.210)

Circumference of column circle (A_p) = $\pi \times D_0$ (3.211)

Width of foundation (b_f) = A_f / A_p (3.212)

Hence inner diameter of base raft (d_i) = $2[D_0/2 - b_f/2]$ (3.213)

Outer diameter (d_o) = $2[D_0/2 + b_f/2]$ (3.214)

Area of annular raft $(A_{ar}) = \frac{\pi}{4}\left(d_0^2 - d_i^2\right)$ (3.215)

Moment of inertia of slab about a diametrical axis $(I_{rs}) = \frac{\pi}{64}\left(d_0^4 - d_i^4\right)$ (3.216)

When tank is empty, total load (W_{27}) = $W_{24} + W_{25}$ (3.217)

Consideration in design of foundation is made for the following two different cases:

3.4.1 For lateral load (earthquake / wind load) condition:

The soil pressure at the edge along a diameter when the tank is full

$$q_{cf} = \frac{W_{26}}{A_{ar}} \pm \frac{M_{bs}}{I_{rs}} \times \frac{d_0}{2} \qquad (3.218)$$

The soil pressure at the edge along a diameter when the tank is empty

$$q_{ce} = \frac{W_{27}}{A_{ar}} \pm \frac{M_{bs}}{I_{rs}} \times \frac{d_0}{2} \qquad (3.219)$$

According to IS code [15] under the wind / earthquake load, the allowable bearing capacity is to be increased to 50%.

If the calculated soil pressure is less than the allowable soil pressure, then the selected foundation area is safe, otherwise, it is to be revised adequately.

3.4.1.1 Design of annular raft slab (Fig33):

Outer radius of raft = r_0

Inner radius of raft = r_i

Mean radius of raft = R_0

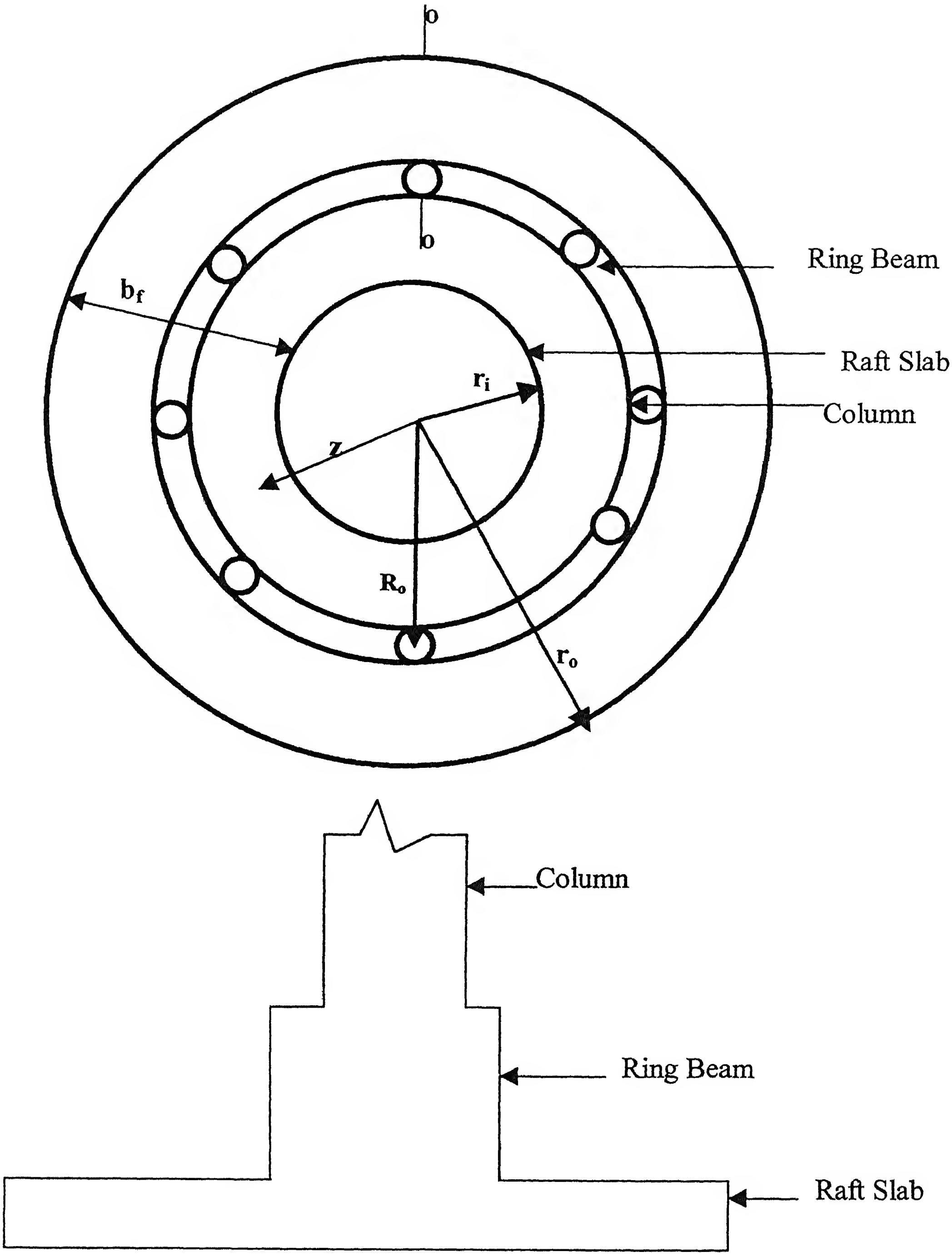
Based on thin plate theory the design moments [18] :

For $z < R_0$

i) Radial moment

$$M_{ro} = \frac{q^1 r_0^2}{4}\left(1 - \frac{r_i^2}{z^2}\right)\left(\log_e \frac{r_0}{R_0} + 0.5 - \frac{R_0^2}{2r_0^2}\right) + \frac{3q^1 z^2}{16} - \frac{q^1 r_i^2}{4}\left[\log_e \frac{z}{r_0} + \frac{3}{4}\left(1 + \frac{r_0^2}{r_i^2} - \frac{r_0^2}{z^2}\right) + \left(\frac{r_0^2 - z^2}{r_o^2 - r_i^2}\right)\left(\frac{r_i}{z}\right)^2 \log_e \frac{r_o}{r_i}\right]$$

(3.220)



Section o - o
Fig 35 Annular Circular Raft Foundation

ii) Circumferential moment

$$M_{\theta o}=\frac{q^1 r_0^2}{4}\left(1+\frac{r_i^2}{z^2}\right)\left(\log_e\frac{r_0}{R_0}+0.5-\frac{R_0^2}{2r_0^2}\right)+\frac{q^1 z^2}{16}-\frac{q^1 r_i^2}{4}\left[\log_e\frac{z}{r_0}+\frac{3}{4}\left(-\frac{1}{3}+\frac{r_0^2}{r_i^2}+\frac{r_0^2}{z^2}\right)-\left(\frac{r_0^2+z^2}{r_o^2-r_i^2}\right)\left(\frac{r_i}{z}\right)^2\log_e\frac{r_o}{r_i}\right]$$

(3.221)

For z > R_0

i) Radial moment

$$M_{r1}=\frac{q^1\left(r_0^2-r_i^2\right)}{4}\left[\log_e\frac{R_0}{z}-\frac{1}{2}+\left(\frac{z^2-r_i^2}{r_0^2-r_i^2}\right)\left(\frac{r_0}{z}\right)^2\lambda+\frac{R_0^2}{2z^2}\right]+\frac{3q^1 z^2}{16}-\frac{q^1 r_i^2}{4}\left[\log_e\frac{z}{r_0}+\frac{3}{4}\left(1+\frac{r_0^2}{r_i^2}-\frac{r_0^2}{z^2}\right)+\left(\frac{r_0^2-z^2}{r_o^2-r_i^2}\right)\left(\frac{r_i}{z}\right)^2\log_e\frac{r_o}{r_i}\right]$$

(3.222)

ii) Circumferential moment

$$M_{\theta 1}=\frac{q^1\left(r_0^2-r_i^2\right)}{4}\left[\log_e\frac{R_0}{z}+\frac{1}{2}+\left(\frac{z^2+r_i^2}{r_0^2-r_i^2}\right)\left(\frac{r_0}{z}\right)^2\lambda-\frac{R_0^2}{2z^2}\right]+\frac{q^1 z^2}{16}-$$
$$\frac{q^1 r_i^2}{4}\left[\log_e\frac{z}{r_0}+\frac{3}{4}\left(-\frac{1}{3}+\frac{r_0^2}{r_i^2}+\frac{r_0^2}{z^2}\right)-\left(\frac{r_0^2+z^2}{r_o^2-r_i^2}\right)\left(\frac{r_i}{z}\right)^2\log_e\frac{r_o}{r_i}\right]$$

(3.223)

Where, $\lambda=\log_e\frac{r_o}{R_0}+\frac{1}{2}-\frac{R_o^2}{2r_o^2}$

$$q^1=\frac{W_{26}}{A_{ar}}+\frac{1}{2}\left(\frac{M_{bs}}{I_{rs}}\times\frac{d_o}{2}\right)$$

The maximum radial and Circumferential moments is to be determined by using the trial method. The depth of the slab will be based on maximum moment (radial and Circumferential).

Maximum bending moment = M_{rs}

$$\text{Depth of slab }(d_s)=\sqrt{\frac{M_{rs}}{1000\times R'}} \qquad (3.224)$$

Maximum area of radial reinforcement $(A_{1f}) = \dfrac{M_{r,\max}}{\sigma_{st} \times j \times d'}$ (3.225)

Spacing of radial reinforcement $(S_{1f}) = \dfrac{1000 \times A_{\phi f}}{A_{1f}}$ (3.226)

Maximum area of circumferential reinforcement $(A_{1cf}) = \dfrac{M_{\theta,\max}}{\sigma_{st} \times j \times d'}$ (3.227)

Spacing of circumferential reinforcement $(S_{1cf}) = \dfrac{1000 \times A_{\phi f}}{A_{2f}}$ (3.228)

3.4.1.2 Design of circular beam of raft:

Design load per meter on raft beam $(W_{28}) = \dfrac{q^1 \times \dfrac{\pi(d_o^2 - d_i^2)}{4}}{\pi D_o}$ (3.229)

Maximum negative bending moment at support $(M_0) = C_1 W_{28} R_0^2 2\theta$ (3.230)

Maximum positive bending moment $(M_c) = C_2 W_{28} R_0^2 2\theta$ (3.231)

Maximum torsional moment $(M_\alpha^t) = C_1 W_{28} R_0^2 2\theta$ (3.232)

Where C_1, C_2, C_3 are the coefficient of bending and twisting moment

$2\theta = 2\pi/n$, where n is the number of columns.

Required depth of beam $(d_{rf}) = \sqrt{\dfrac{M}{Rb_{rf}}}$ (3.233)

Where M is the maximum bending moment.

Maximum shear force at support $(F_0) = W_{28} R_0 \theta$ (3.234)

Shear force at any point $(F) = W_{28} R_0 (\theta - \alpha)$ (3.235)

Shear force at the point of maximum torsional moment $(F_m) = W_{28} R_0 (\theta - \alpha_m)$ (3.236)

Bending moment at the point of maximum torsional moment

$$M\alpha_m \text{ (sagging)} = W_{28} R_0^2 (\theta \sin\alpha_m + \theta \cot\theta \cos\alpha_m - 1) \qquad (3.237)$$

The torsional moment at any point

$$M^t\alpha = W_{28} R_0^2 \{\theta \cos\alpha - \theta \cot\theta \sin\alpha - (\theta - \alpha)\} \qquad (3.238)$$

At the support $\alpha = 0$, $M^t_0 = 0$

At the mid-span $\alpha = \theta$, $M^t_c = 0$

Hence the following combination of bending moment and torsional moment are obtained:

(a)At the point of maximum torsion:

Bending moment $= M_\alpha$ and Torsional moment $= M^t_{\alpha m}$

(b) At the support: Bending moment $= M_0$ and Torsional moment $= M^t_0$

(c) At mid span: Bending moment $= M_c$ and Torsional moment $= M^t_c$

The c/s area required for the main reinforcement is obtained with the consideration of the above three conditions.

(i) Main Reinforcement

(a) Section at point of maximum torsion

$T = M^t_{max}$, $M_\alpha = M$

$$\text{Equivalent bending moment } (M_{el}) = M + T\left[\frac{1+\frac{d_{rf}}{b_{rf}}}{1.7}\right] \qquad (3.239)$$

$$\text{Area of steel } (A_{2f}) = M_{e\,1}/(\sigma_{st}\, j\, d_{rf}) \qquad (3.240)$$

$$\text{Numbers of bars } (N_{1f}) = A_{1f}/A_{\phi f1}$$

If $T > M$, the compression reinforcements are to be provided.

$$\text{Equivalent bending moment}(M_{e2}) = T\left[\frac{1+\frac{d_{rf}}{b_{rf}}}{1.7}\right] - M \qquad (3.241)$$

$$\text{Compression reinforcement } (A_{3f}) = M_{e2}/(\sigma_{st}\, j\, d_{rf}) \qquad (3.242)$$

$$\text{Numbers of bars } (N_{2f}) = A_{3f}/A_{\phi f2}$$

(b) Section at maximum hogging bending moment (support):

$M_0 = M_{max}$, $M^t_0 = 0$

$$\text{Area of steel } (A_{4f}) = M_0/(\sigma_{st}\, j\, d_{rf}) \qquad (3.243)$$

$$\text{Numbers of bars } (N_{3f}) = A_{4f}/A_{\phi f3}$$

(c) Section at maximum sagging bending moment (mid-span):

Maximum bending moment at center= M_c

$$\text{Torsional moment } (M^t_c) = 0$$

$$\text{Area of steel } (A_{5f}) = M_c/(\sigma_{st}\, j\, d_{rf}), \qquad (3.244)$$

$$\text{Numbers of bars } (N_{4f}) = A_{5f}/A_{\phi f4} \qquad (3.245)$$

(ii) Transverse reinforcement:

(a) At point of maximum torsional moment:

At the point of maximum torsion shear force (V) = F_m

$$\text{Equivalent shear force } (V_e) = V + 1.6\, T/b_{rf} \qquad (3.246)$$

$$\text{Shear stress } (\tau_{ve}) = V_e/b_{rf}\, d_{rf} \qquad (3.247)$$

$\tau_{v\,e}$ should be always less than $\tau_{c\,max}$ otherwise the section is to be redesign [16].

If $\tau_{v\,e} > \tau_c$, shear reinforcement is necessary, otherwise minimum shear reinforcements are to be provided [16].

Cross-section area of the stirrups [16]

$$A_{sv} = \frac{T \times S_{v}}{b^1 d^1 \sigma_{sv}} + \frac{V \times S_v}{2.5 d^1 \sigma_{sv}} \tag{3.248}$$

Where b^1 = center- to- center distance between corner bars in the direction of the width.

d^1 = center- to- center distance between corner bars in the direction of depth.

But the total reinforcement shall not be less than $(\tau_{ve} - \tau_c)\, b_2\, S_v/\sigma_{sv}$

The spacing of the stirrups shall not exceed the least of x_1, $(x_1+y_1)/4$, and 300 mm, where x_1 and y_1 are the respectively the shorter and long dimension of the stirrup.

(b) At the point of maximum shear (supports):

Shear stress $\tau_v = F_0 / b_{rf}\, d_{rf}$

τ_v should be always less than $\tau_{c\,max}$, otherwise the section is to be redesigned [16]

If $\tau_v > \tau_c$, then shear reinforcement is necessary, otherwise minimum shear reinforcements are to be provided [16].

$$V_c = \tau_c \times b_{rf}\, d_{rf}$$

Design shear force $(V_s) = F_0 - V_c$ (3.249)

The spacing of stirrups $(S_{v1}) = (\sigma_{sv} \times A_{sv1} \times d_{rf})/V_s$ (3.250)

Where A_{sv1}= area of 4-leggd stirrups.

(c)At mid-span: At the mid-span, shear force is zero. Hence minimum/nominal reinforcement are to be provided.

Minimum shear reinforcement in the form of stirrups shall be provided such that [16]

$$\frac{A_{sv2}}{b_{rf} S_{v2}} \geq \frac{0.4}{0.87 f_y} \tag{3.251}$$

Where A_{sv2} = total cross-section area of stirrups legs effective in shear,

S_{v2}= stirrups spacing along the length of the member,

b_2= breadth of the beam or breadth of the web of flanged beam, and

f_y = characteristic strength of the stirrup reinforcement in N/mm^2 which shall not be taken greater than 415 N/mm^2.

Maximum spacing of shear reinforcement measured along the axis of the member shall not exceed 0.75 d. In no case shall the spacing exceed 300mm.

iii) Side face reinforcement: where the depth of the web in a beam exceeds 450 mm, side face reinforcement shall be provided along the two faces. The total area of such reinforcement shall be not less than 0.2 percent of the web area and shall be distributed equally on two faces at spacing not exceeding 300mm or web thickness whichever is less [16].

$$A_{sd} = (0.2 \times b_{rf} \times d_{rf})/100 \text{ mm} \tag{3.252}$$

3.4.2 without considering lateral load condition:

The soil pressure at the edge along a diameter when the tank is full

$$q_{cf1} = \frac{W_{26}}{A_{ar}} \tag{3.253}$$

The soil pressure at the edge along a diameter when the tank is empty

$$q_{ce1} = \frac{W_{27}}{A_{ar}} \tag{3.254}$$

If the calculated soil pressure is less than the allowable soil pressure, then the selected foundation area is safe, otherwise, it is to be revised adequately.

The maximum radial and Circumferential moments is to be determined by using the trial method. The depth of the slab will be based on maximum moment (radial and Circumferential).

Maximum bending moment = M_{rs1}

$$\text{Depth of slab } (d_{s1}) = \sqrt{\frac{M_{rs1}}{1000 \times R'}} \tag{3.258}$$

$$\text{Maximum area of radial reinforcement } (A_{1f1}) = \frac{M_{r1,\max}}{\sigma_{st} \times j \times d'} \tag{3.259}$$

$$\text{Spacing of radial reinforcement } (S_{1f1}) = \frac{1000 \times A_{\phi f}}{A_{1f}} \tag{3.260}$$

$$\text{Maximum area of circumferential reinforcement } (A_{1cf1}) = \frac{M_{\theta 1,\max}}{\sigma_{st} \times j \times d'} \tag{3.261}$$

$$\text{Spacing of circumferential reinforcement } (S_{1cf1}) = \frac{1000 \times A_{\phi f}}{A_{2f}} \tag{3.262}$$

3.4.2.2 Design of circular beam of raft:

$$\text{Design load per meter on raft beam } (W^1{}_{28}) = \frac{q_1 \times \dfrac{\pi\left(d_o^2 - d_i^2\right)}{4}}{\pi D_o} \tag{3.263}$$

$$\text{Maximum negative bending moment at support } (M_0) = C_1\ W^1{}_{28}\ R^2{}_0\ 2\theta \tag{3.264}$$

$$\text{Maximum positive bending moment } (M_c) = C_2\ W^1{}_{28}\ R^2{}_0\ 2\theta \tag{3.265}$$

$$\text{Maximum torsional moment } (M^t{}_\alpha) = C_1\ W^1{}_{28} R^2{}_0\ 2\theta \tag{3.266}$$

Where C_1, C_2,C_3 are the coefficient of bending and twisting moment

$2\theta = 2\pi/n$, where n is the number of columns.

$$\text{Required depth of beam } (d_{rf1}) = \sqrt{\frac{M_1}{Rb_{rf}}} \tag{3.267}$$

Where M_1 is the maximum bending moment.

$$\text{Maximum shear force at support } (F_0) = W^1{}_{28} R_0 \theta \tag{3.268}$$

$$\text{Shear force at any point } (F) = W^1{}_{28} R_0 (\theta - \alpha) \tag{3.269}$$

$$\text{Shear force at the point of maximum torsional moment } (F_m) = W^1{}_{28} R_0 (\theta - \alpha_m) \tag{3.270}$$

Bending moment at the point of maximum torsional moment

$$M\alpha_m \text{ (sagging)} = W^1{}_{28} R^2{}_0 (\theta \sin\alpha_m + \theta \operatorname{Cot}\theta \operatorname{Cos}\alpha_m - 1) \tag{3.271}$$

The torsional moment at any point

$$M^t\alpha = W^1{}_{28} R^2{}_0 \{\theta \operatorname{Cos}\alpha - \theta \operatorname{Cot}\theta \operatorname{Sin}\alpha - (\theta - \alpha)\} \tag{3.272}$$

At the support $\alpha = 0$, $M^t{}_0 = 0$

At the mid-span $\alpha = \theta$, $M^t{}_c = 0$

Hence the following combination of bending moment and torsional moment are obtained:

(a)At the point of maximum torsion:

Bending moment $= M_\alpha$ and Torsional moment $= M^t{}_{\alpha m}$

(b) At the support: Bending moment $= M_0$ and Torsional moment $= M^t{}_0$

(c) At mid span: Bending moment $= M_c$ and Torsional moment $= M^t{}_c$

The c/s area required for the main reinforcement is obtained with the consideration of the above three conditions.

(i) Main Reinforcement

(a) Section at point of maximum torsion

$T = M^t_{max}$, $M_\alpha = M$

$$\text{Equivalent bending moment } (M_{e1}) = M + T\left[\frac{1+\frac{d_{rf}}{b_{rf}}}{1.7}\right]$$

Area of steel (A_{2f}) = $M_{e1}/(\sigma_{st}\, j\, d_{rf})$

Numbers of bars (N_{1f}) = $A_{1f}/A_{\phi f1}$

If T > M, the compression reinforcements are to be provided.

$$\text{Equivalent bending moment} (M_{e2}) = T\left[\frac{1+\frac{d_{rf}}{b_{rf}}}{1.7}\right] - M$$

Compression reinforcement (A_{3f}) = $M_{e2}/(\sigma_{st}\, j\, d_{rf})$

Numbers of bars (N_{2f}) = $A_{3f}/A_{\phi f2}$

(b) Section at maximum hogging bending moment (support):

$M_0 = M_{max}$, $M^t_0 = 0$

Area of steel (A_{4f}) = $M_0/(\sigma_{st}\, j\, d_{rf})$

Numbers of bars (N_{3f}) = $A_{4f}/A_{\phi f3}$

(c) Section at maximum sagging bending moment (mid-span):

Maximum bending moment at center= M_c

Torsional moment (M^t_c) = 0

Area of steel (A_{5f}) = $M_c/(\sigma_{st}\, j\, d_{rf})$,

Numbers of bars (N_{4f}) = $A_{5f}/A_{\phi f4}$

(ii) Transverse reinforcement:

(a) At point of maximum torsional moment:

At the point of maximum torsion shear force (V) = F_m

Equivalent shear force (V_e) = V + 1.6 T/ b_{rf}

Shear stress ($\tau_{v\,e}$) = $V_e/b_{rf}\,d_{rf}$

$\tau_{v\,e}$ should be always less than $\tau_{c\,max}$ otherwise the section is to be redesign [16].

If $\tau_{v\,e} > \tau_c$, shear reinforcement is necessary, otherwise minimum shear reinforcements are to be provided [16].

Cross-section area of the stirrups [16]

$$A_{sv}=\frac{T\times S_v}{b^1 d^1 \sigma_{sv}}+\frac{V\times S_v}{2.5 d^1 \sigma_{sv}}$$

Where b^1 = center- to- center distance between corner bars in the direction of the width.

d^1 = center- to- center distance between corner bars in the direction of depth.

But the total reinforcement shall not be less than $(\tau_{v\,e} - \tau_c)\,b_2\,S_v/\sigma_{sv}$

The spacing of the stirrups shall not exceed the least of x_1, $(x_1+y_1)/4$, and 300 mm, where x_1 and y_1 are the respectively the shorter and long dimension of the stirrup.

(b) At the point of maximum shear (supports):

Shear stress $\tau_v = F_0 / b_{rf}\,d_{rf}$

τ_v should be always less than $\tau_{c\,max}$, otherwise the section is to be redesigned [16]

If $\tau_v > \tau_c$, then shear reinforcement is necessary, otherwise minimum shear reinforcements are to be provided [16].

$V_c = \tau_c \times b_{rf}\,d_{rf}$

Design shear force (V_s) = $F_0 - V_c$

The spacing of stirrups (S_{v1}) = ($\sigma_{sv} \times A_{sv1} \times d_{rf}$)/$V_s$

Where A_{sv1}= area of 4-leggd stirrups.

(c)At mid-span: At the mid-span, shear force is zero. Hence minimum/nominal reinforcement are to be provided.

Minimum shear reinforcement in the form of stirrups shall be provided such that [16]

$$\frac{A_{sv2}}{b_{rf} S_{v2}} \geq \frac{0.4}{0.87 f_y}$$

Where A_{sv2} = total cross-section area of stirrups legs effective in shear,

S_{v2}= stirrups spacing along the length of the member,

b_2= breadth of the beam or breadth of the web of flanged beam, and

f_y = characteristic strength of the stirrup reinforcement in N/mm^2 which shall not be taken greater than 415 N/mm^2.

Maximum spacing of shear reinforcement measured along the axis of the member shall not exceed 0.75 d. In no case shall the spacing exceed 300mm.

Chapter 4

Algorithm for the Design of Water Tank

4.1 Introduction

Design process of any structure require step by step procedure, which could be represented in the form of an algorithm for better understanding the whole procedure involved in the design, and also in developing computer program code (software). Design of water tank is done in three different stages. In the first stage, the design for the container is made. Then in the second stage, design for staging is done. Finally, foundation is designed after getting all gravity loads and lateral loads acting on the structure. In this study, the algorithm is developed for the Intze water tank supported on staging, foundation on an annular raft-type foundation.

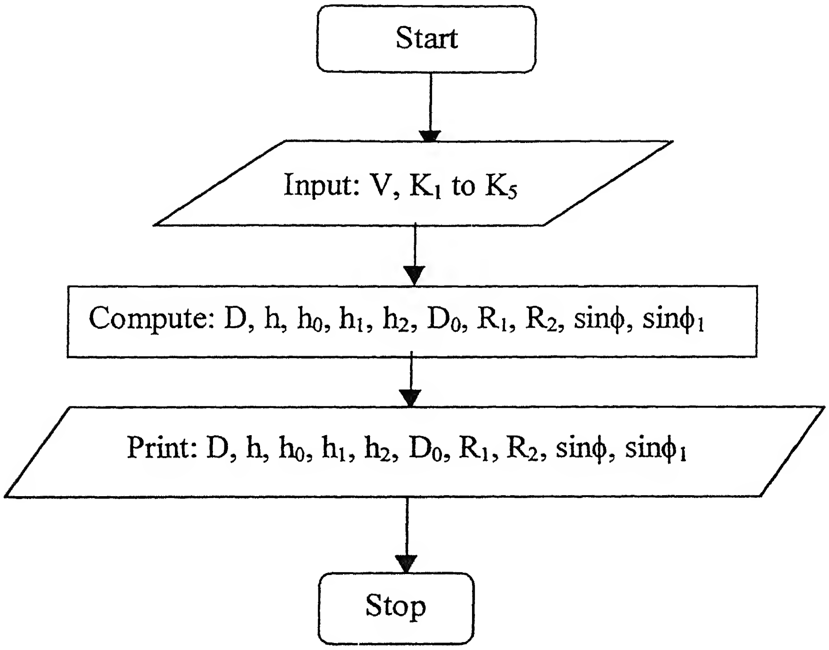
4.2 Algorithm for the Container Design

The algorithm for the container design is developed keeping in view the sequences that one involved starting from preliminary design for selecting of the different dimensions followed by the steps for analysis and final design.

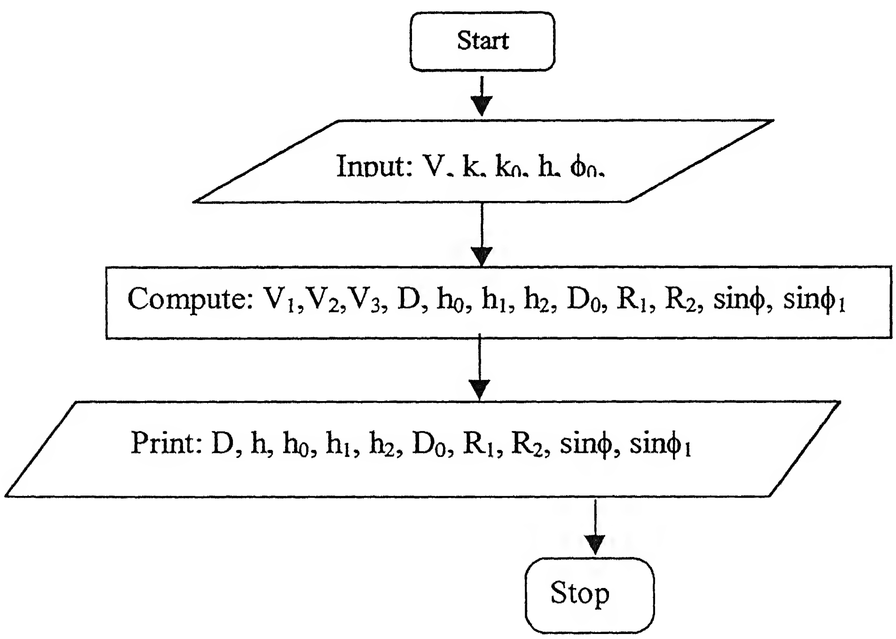
4.2.1 Selection of Container Dimensions

The preliminary selection of container dimensions can be made using the methodologies presented in the Chapter 2. In the present study options are kept in the algorithm, for selection of dimension by the designer.

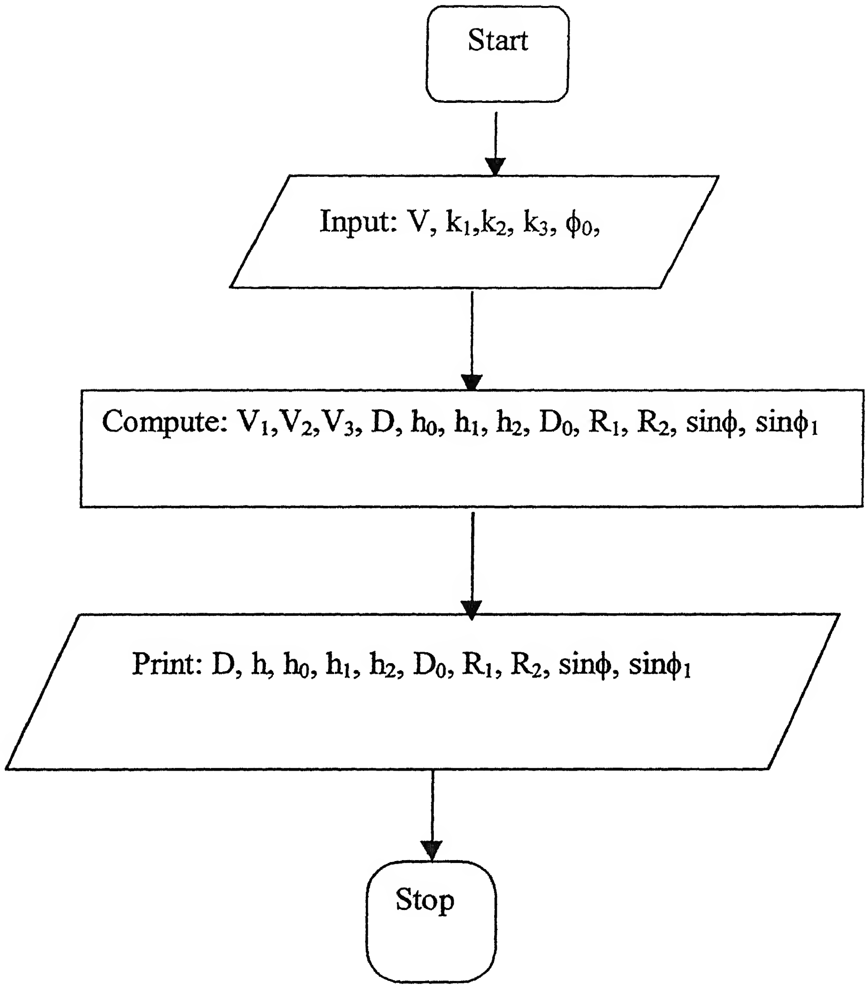
4.2.1.1 Flow chart using the empirical formula suggested by Rao [1]



4.2.1.2 Flow chart according to Jain & Jai Krishna [7]:

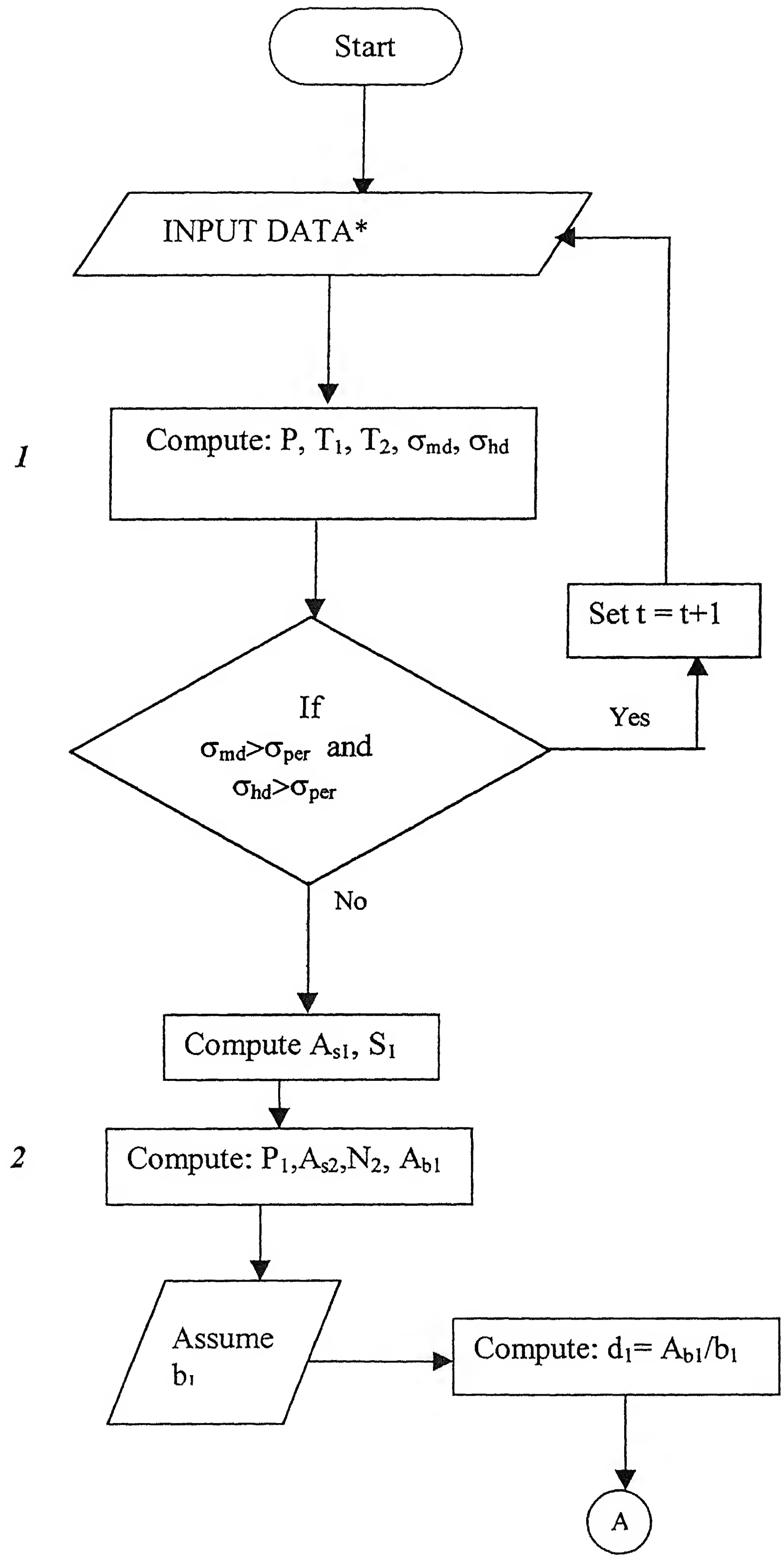


4.2.1.3 Flow chart according to U.P.R.N.L. [8]



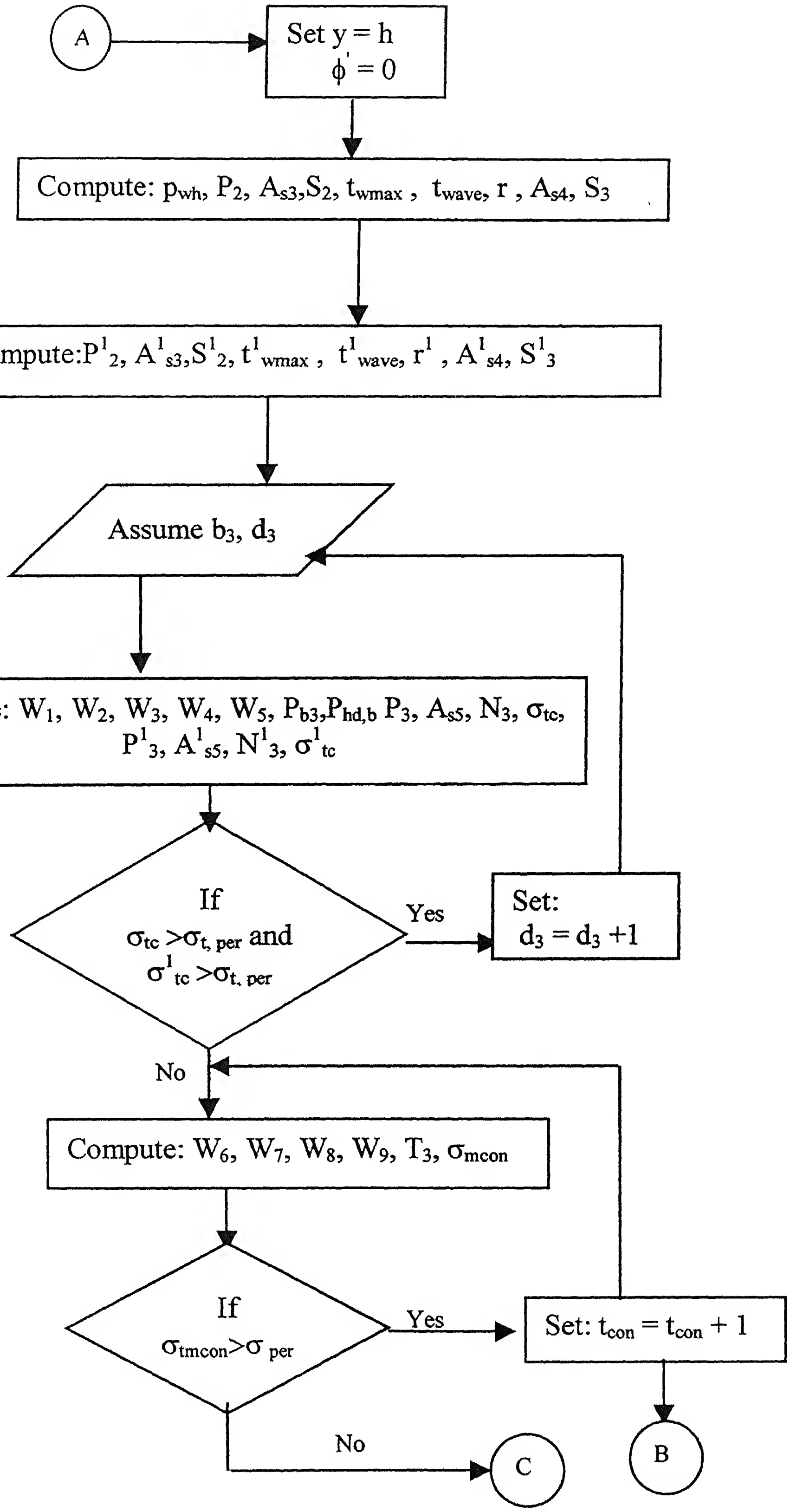
4.2.2 Computation for Stresses and Reinforcements

After obtaining the dimensions of container, the next step is to compute the stresses and reinforcements of the container. The stresses and reinforcements of the container are computed using membrane and continuity analysis and the design is completed with due checks for the permissible stress limits. The flow chart is explained in detail below:

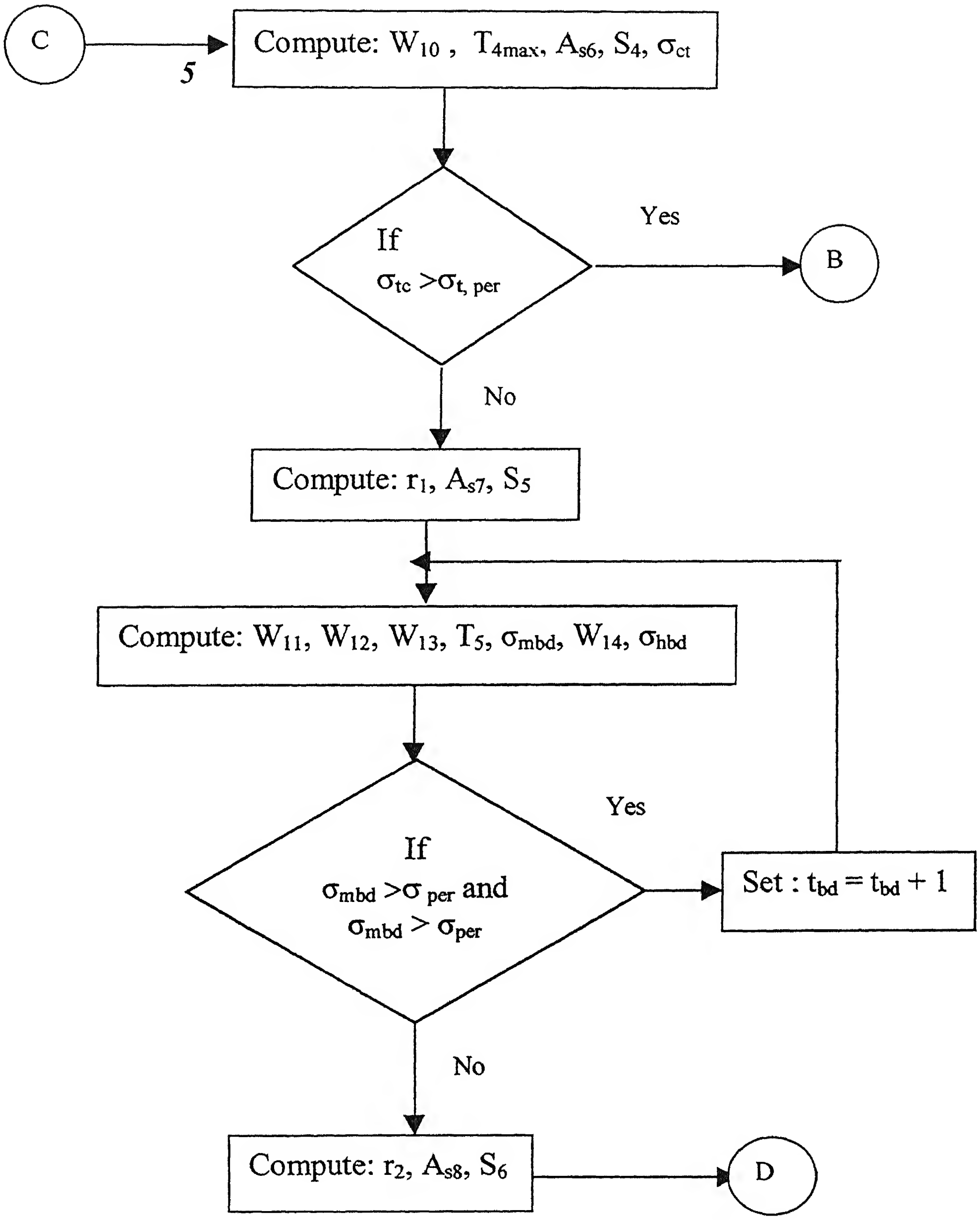

Start
INPUT DATA*
1
Compute: P, T_1, T_2, σ_{md}, σ_{hd}
Set t = t+1
If
$\sigma_{md}>\sigma_{per}$ and
$\sigma_{hd}>\sigma_{per}$
Yes
No
Compute A_{s1}, S_1
2
Compute: P_1,A_{s2},N_2, A_{b1}
Assume
b_1
Compute: d_1= A_{b1}/b_1
A

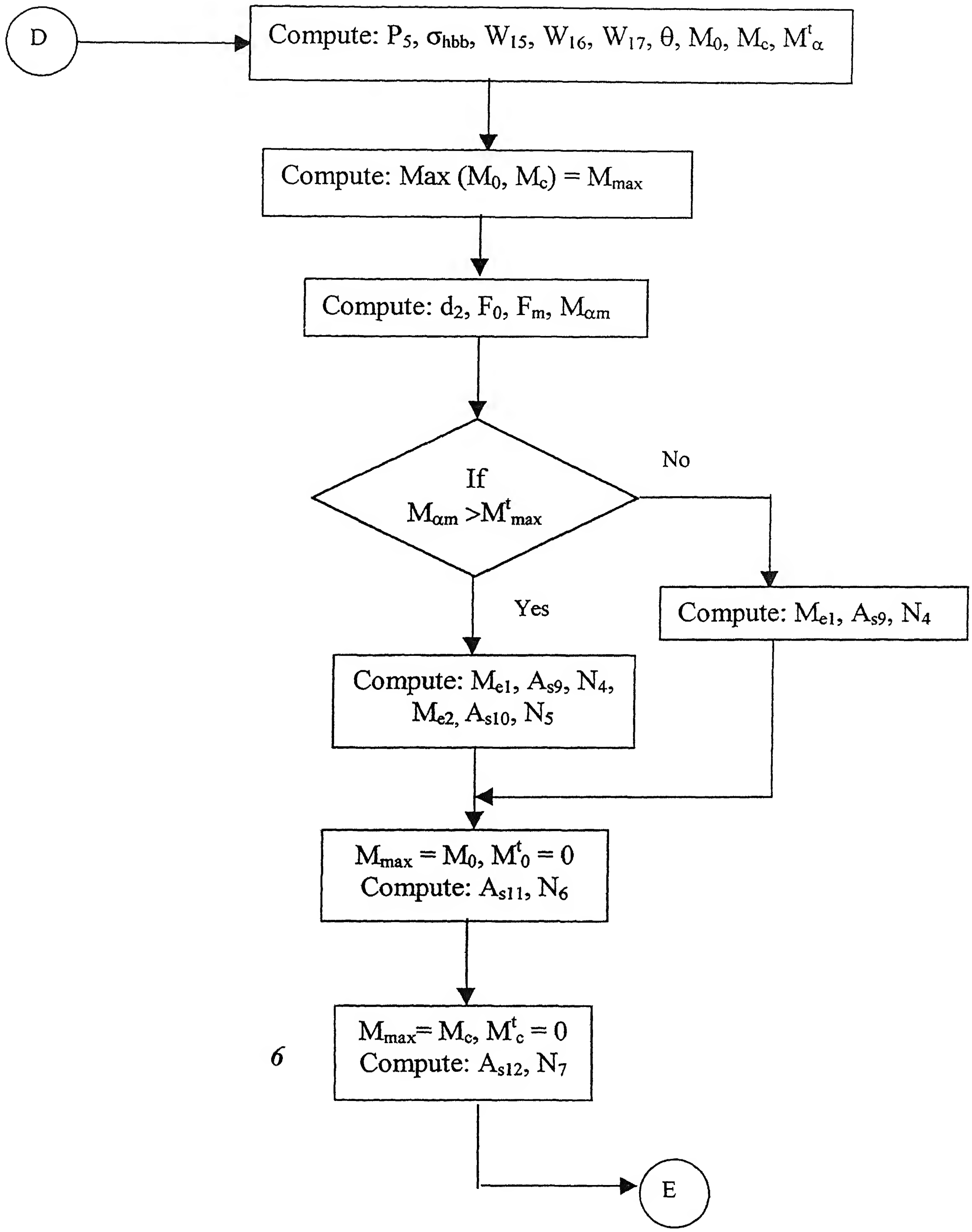
3

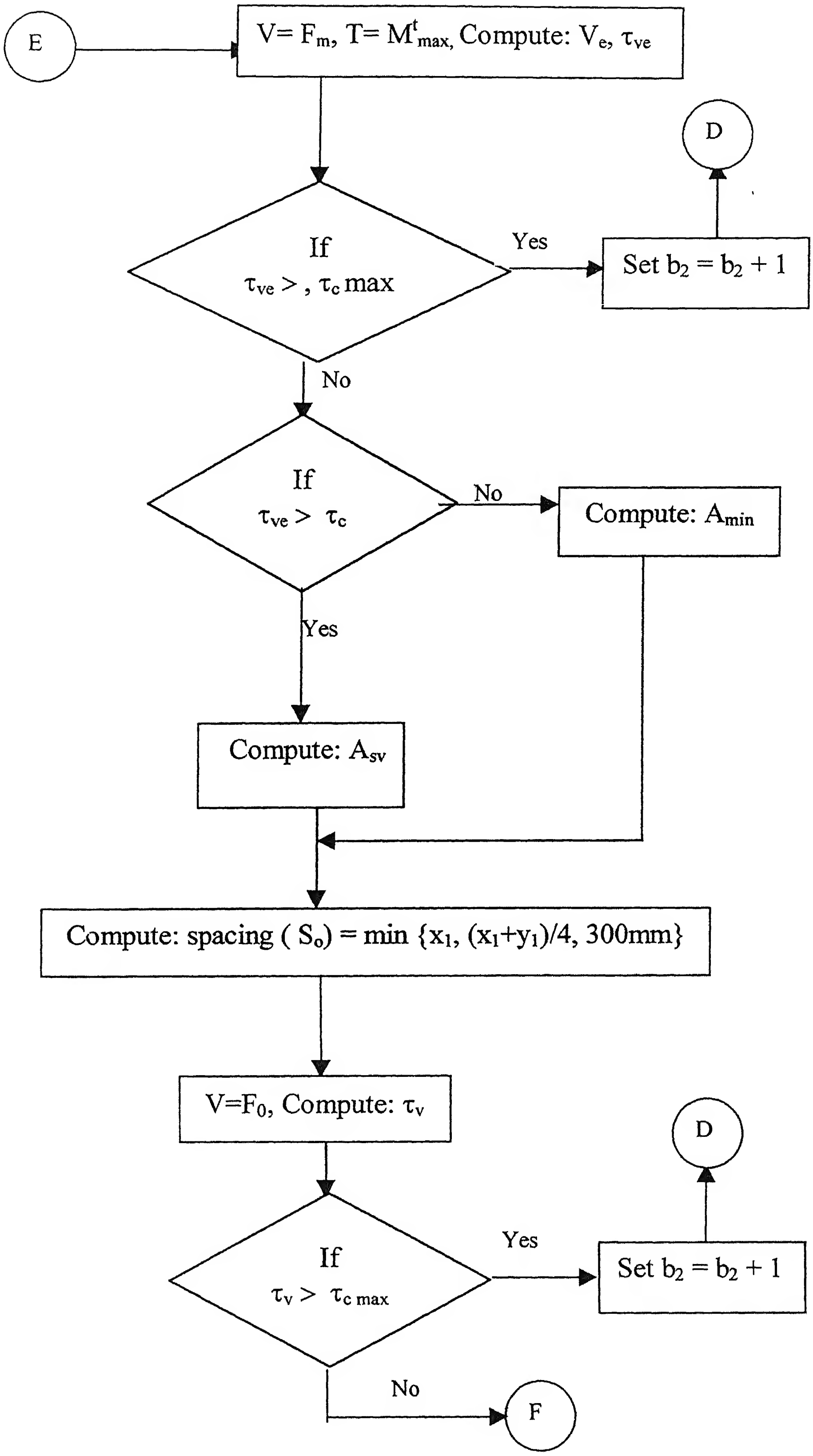
4

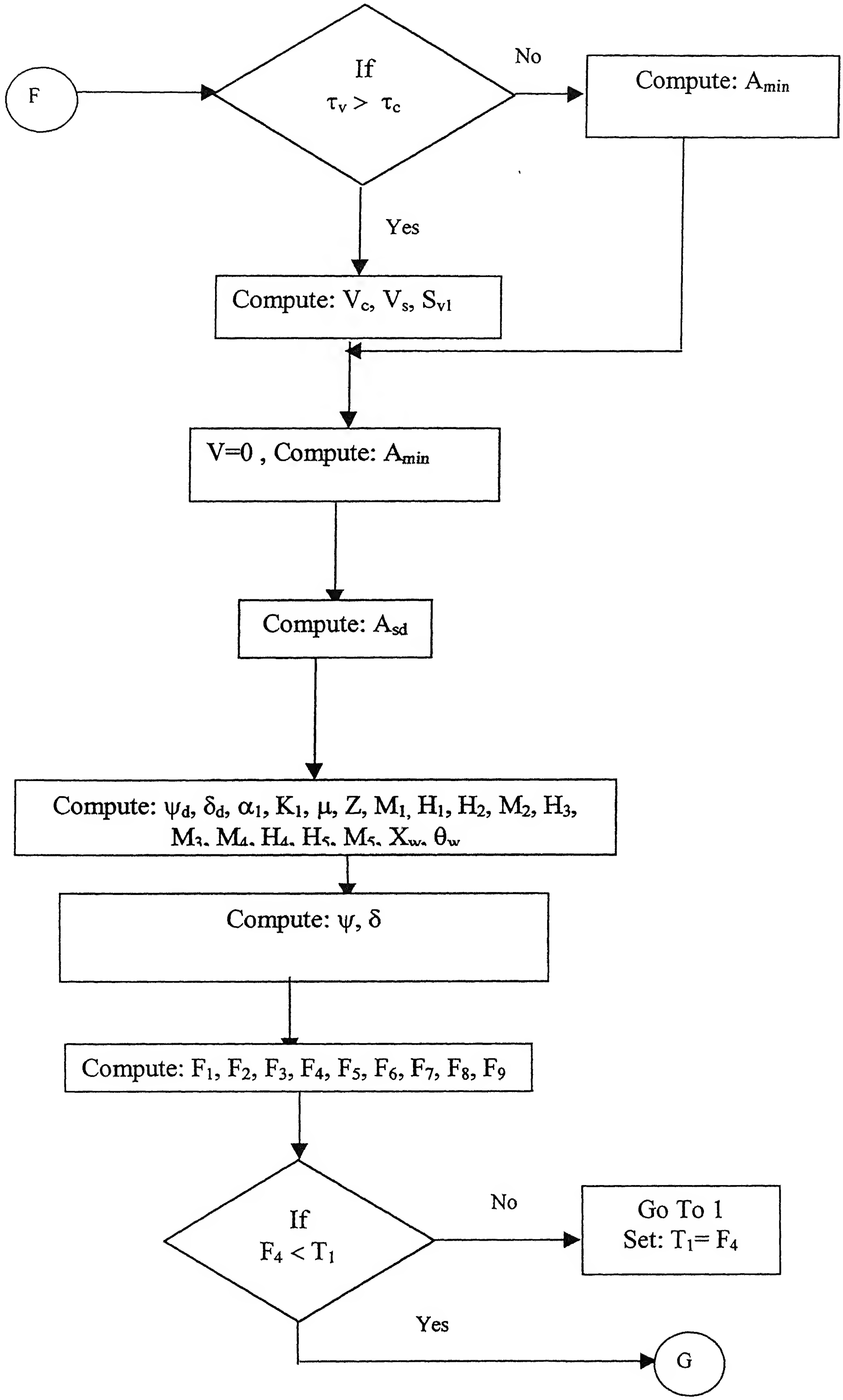


A
Set $y = h$
$\phi' = 0$
Compute: p_{wh}, P_2, A_{s3}, S_2, t_{wmax}, t_{wave}, r, A_{s4}, S_3
Compute: P^1_2, A^1_{s3}, S^1_2, t^1_{wmax}, t^1_{wave}, r^1, A^1_{s4}, S^1_3
Assume b_3, d_3
Compute: W_1, W_2, W_3, W_4, W_5, P_{b3}, $P_{hd,b}$ P_3, A_{s5}, N_3, σ_{tc}, P^1_3, A^1_{s5}, N^1_3, σ^1_{tc}
If $\sigma_{tc} > \sigma_{t,\,per}$ and $\sigma^1_{tc} > \sigma_{t,\,per}$
Yes
Set: $d_3 = d_3 + 1$
No
Compute: W_6, W_7, W_8, W_9, T_3, σ_{mcon}
If $\sigma_{tmcon} > \sigma_{per}$
Yes
Set: $t_{con} = t_{con} + 1$
No
C
B

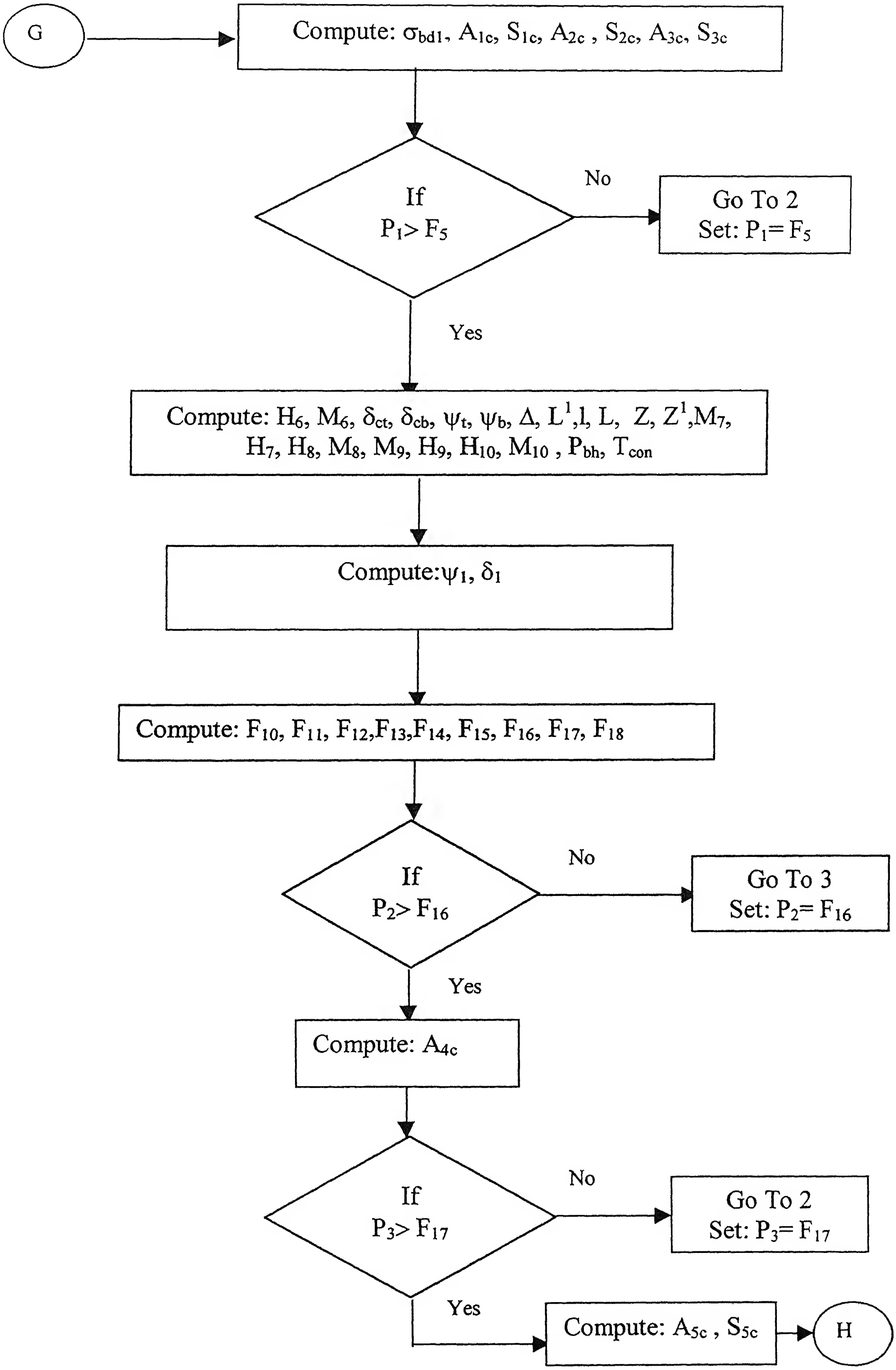

C
5
Compute: W_{10} , T_{4max}, A_{s6}, S_4, σ_{ct}
If
$\sigma_{tc} > \sigma_{t,\ per}$
Yes
B
No
Compute: r_1, A_{s7}, S_5
Compute: W_{11}, W_{12}, W_{13}, T_5, σ_{mbd}, W_{14}, σ_{hbd}
If
$\sigma_{mbd} > \sigma_{per}$ and
$\sigma_{mbd} > \sigma_{per}$
Yes
Set : $t_{bd} = t_{bd} + 1$
No
Compute: r_2, A_{s8}, S_6
D

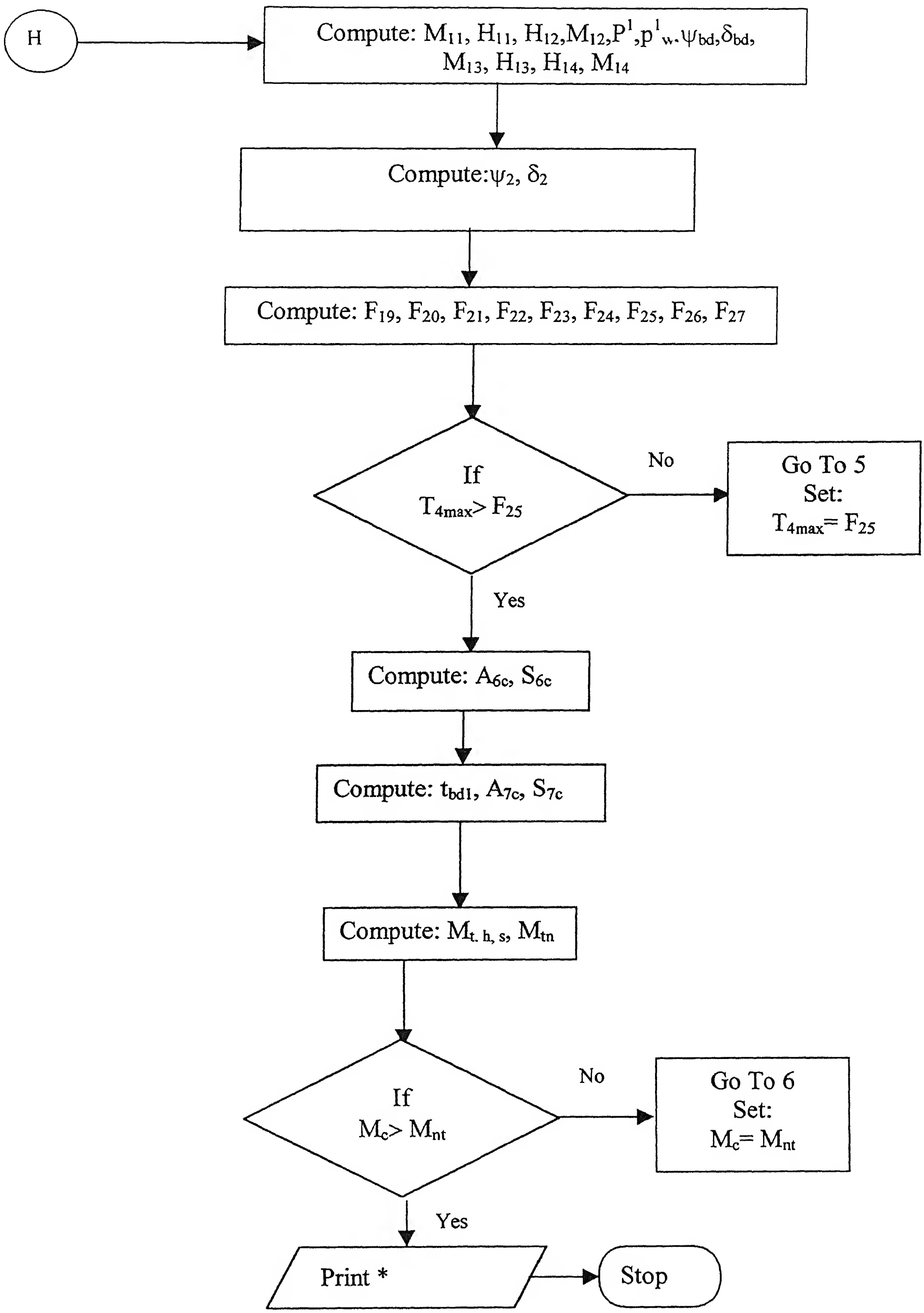

D
Compute: P_5, σ_{hbb}, W_{15}, W_{16}, W_{17}, θ, M_0, M_c, M^t_α
Compute: Max (M_0, M_c) = M_{max}
Compute: d_2, F_0, F_m, $M_{\alpha m}$
If $M_{\alpha m} > M^t_{max}$
No
Yes
Compute: M_{e1}, A_{s9}, N_4
Compute: M_{e1}, A_{s9}, N_4, M_{e2}, A_{s10}, N_5
$M_{max} = M_0$, $M^t_0 = 0$
Compute: A_{s11}, N_6
6
$M_{max} = M_c$, $M^t_c = 0$
Compute: A_{s12}, N_7
E


E
V= F_m, T= M^t_{max}, Compute: V_e, τ_{ve}
D
If
τ_{ve} > , τ_c max
Yes
Set $b_2 = b_2 + 1$
No
If
$\tau_{ve} > \tau_c$
No
Compute: A_{min}
Yes
Compute: A_{sv}
Compute: spacing (S_o) = min {x_1, (x_1+y_1)/4, 300mm}
V=F_0, Compute: τ_v
D
If
$\tau_v > \tau_{c\,max}$
Yes
Set $b_2 = b_2 + 1$
No
F



F
If
$\tau_v > \tau_c$
No
Compute: A_{min}
Yes
Compute: V_c, V_s, S_{v1}
V=0 , Compute: A_{min}
Compute: A_{sd}
Compute: ψ_d, δ_d, α_1, K_1, μ, Z, M_1, H_1, H_2, M_2, H_3, M_3, M_4, H_4, H_5, M_5, X_w, θ_w
Compute: ψ, δ
Compute: F_1, F_2, F_3, F_4, F_5, F_6, F_7, F_8, F_9
If
$F_4 < T_1$
No
Go To 1
Set: $T_1 = F_4$
Yes
G


G
Compute: σ_{bd1}, A_{1c}, S_{1c}, A_{2c} , S_{2c}, A_{3c}, S_{3c}
If
$P_1 > F_5$
No
Go To 2
Set: $P_1 = F_5$
Yes
Compute: H_6, M_6, δ_{ct}, δ_{cb}, ψ_t, ψ_b, Δ, L^1,l, L, Z, Z^1,M_7, H_7, H_8, M_8, M_9, H_9, H_{10}, M_{10} , P_{bh}, T_{con}
Compute:ψ_1, δ_1
Compute: F_{10}, F_{11}, F_{12},F_{13},F_{14}, F_{15}, F_{16}, F_{17}, F_{18}
If
$P_2 > F_{16}$
No
Go To 3
Set: $P_2 = F_{16}$
Yes
Compute: A_{4c}
If
$P_3 > F_{17}$
No
Go To 2
Set: $P_3 = F_{17}$
Yes
Compute: A_{5c} , S_{5c}
H

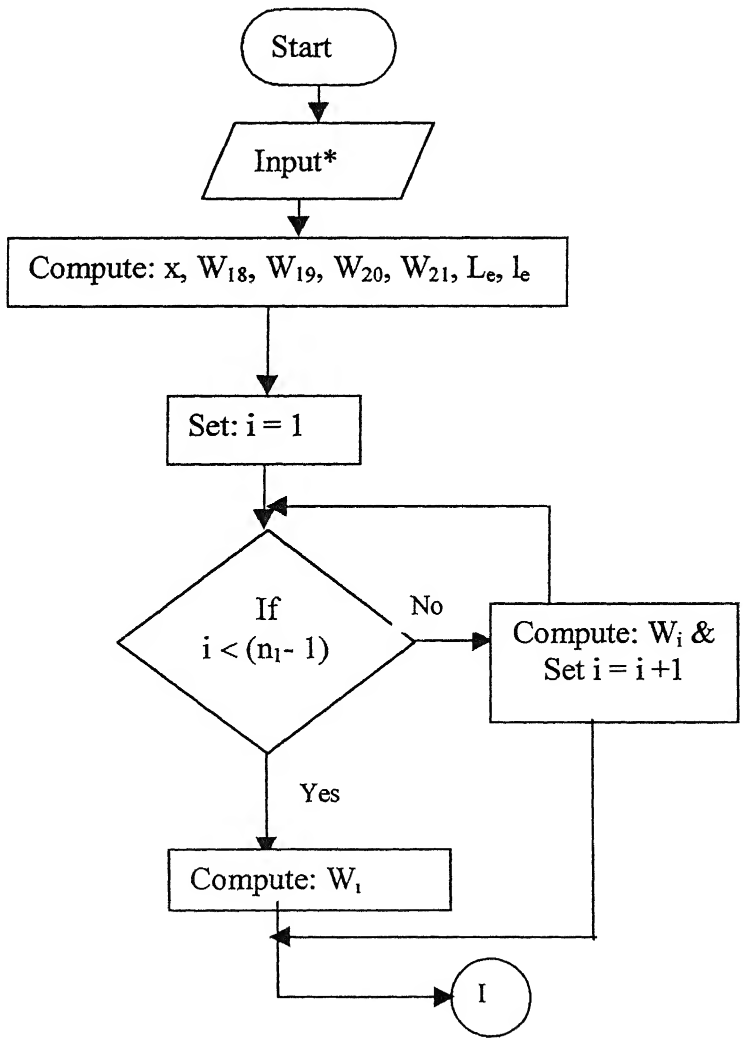

H
Compute: M_{11}, H_{11}, H_{12}, M_{12}, P^1, $p^1{}_w$, ψ_{bd}, δ_{bd}, M_{13}, H_{13}, H_{14}, M_{14}
Compute: ψ_2, δ_2
Compute: F_{19}, F_{20}, F_{21}, F_{22}, F_{23}, F_{24}, F_{25}, F_{26}, F_{27}
If $T_{4max} > F_{25}$
No
Go To 5 Set: $T_{4max} = F_{25}$
Yes
Compute: A_{6c}, S_{6c}
Compute: t_{bd1}, A_{7c}, S_{7c}
Compute: $M_{t.h,s}$, M_{tn}
If $M_c > M_{nt}$
No
Go To 6 Set: $M_c = M_{nt}$
Yes
Print *
Stop

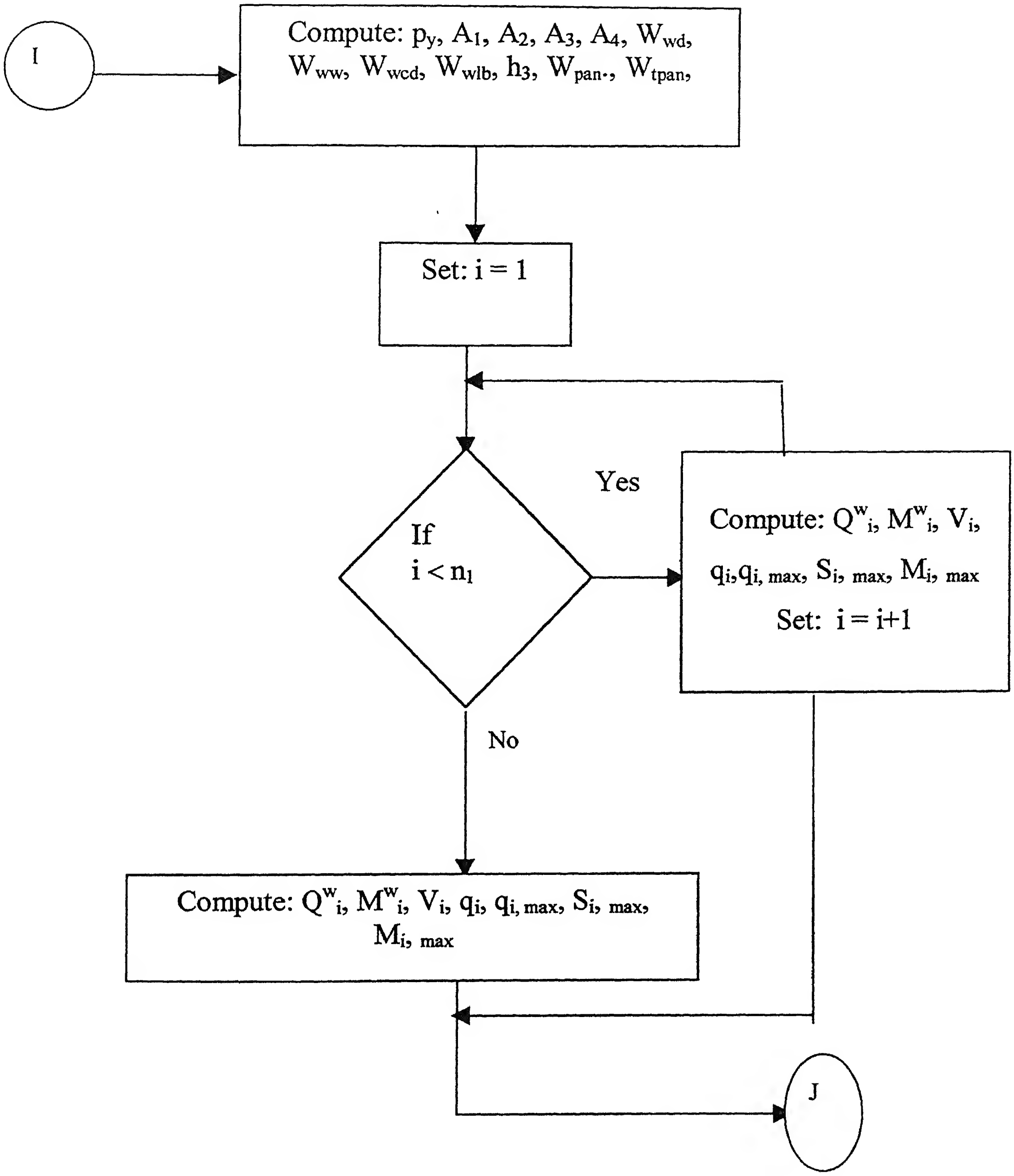
***Input:** Out put from section (4.2.1), t, L.L, γ_c, σ_{per}, A_Φ, σ_{st}, σ_{tc}, m, b_1, w, $t_{w,\ min}$, b_3, d_3, t_{con}, t_{bd}, b_2, d_2, C_1, C_2, C_3, τ_c, $\tau_{c,max}$, E, M_{bal},

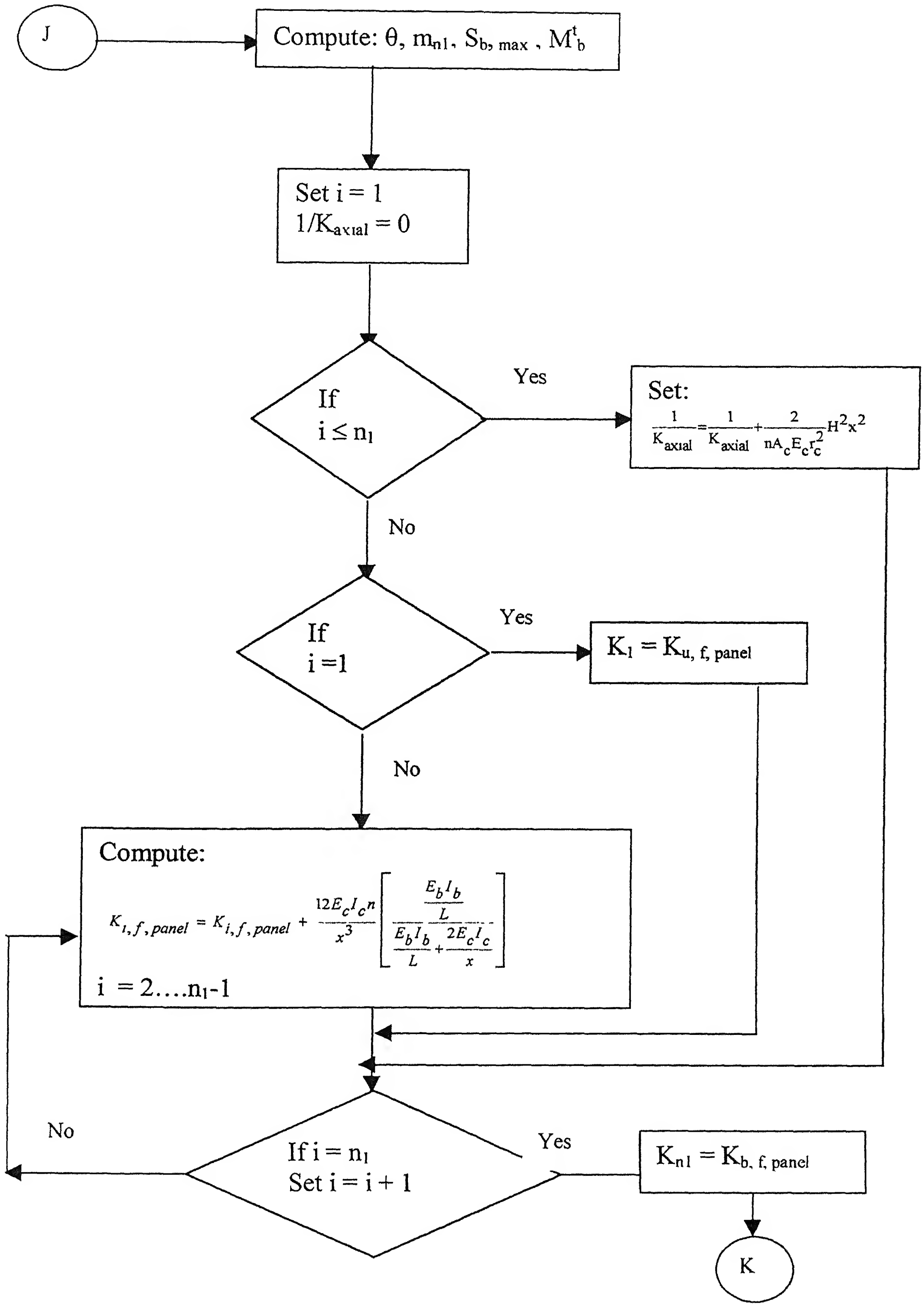
* **Print:** All forces in memebers, Area of reinforcements, Spacing of reinforcements, Dimension of members.

4.3 Algorithm for staging design

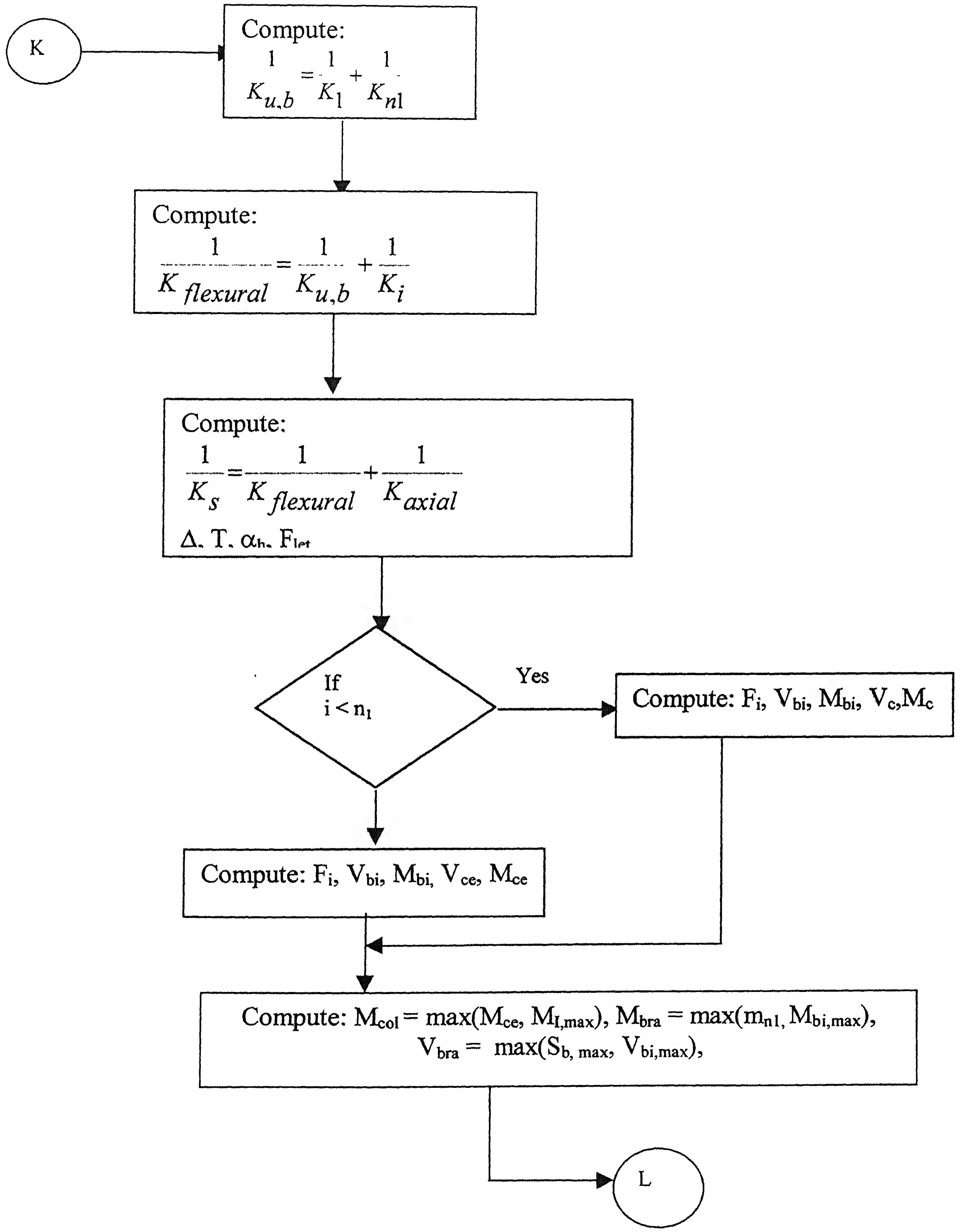
In this section the algorithm for the design of staging is presented. The columns and braces are designed for both gravity and lateral loads, and are also checked for safety. The flowchart is explained in detail below.

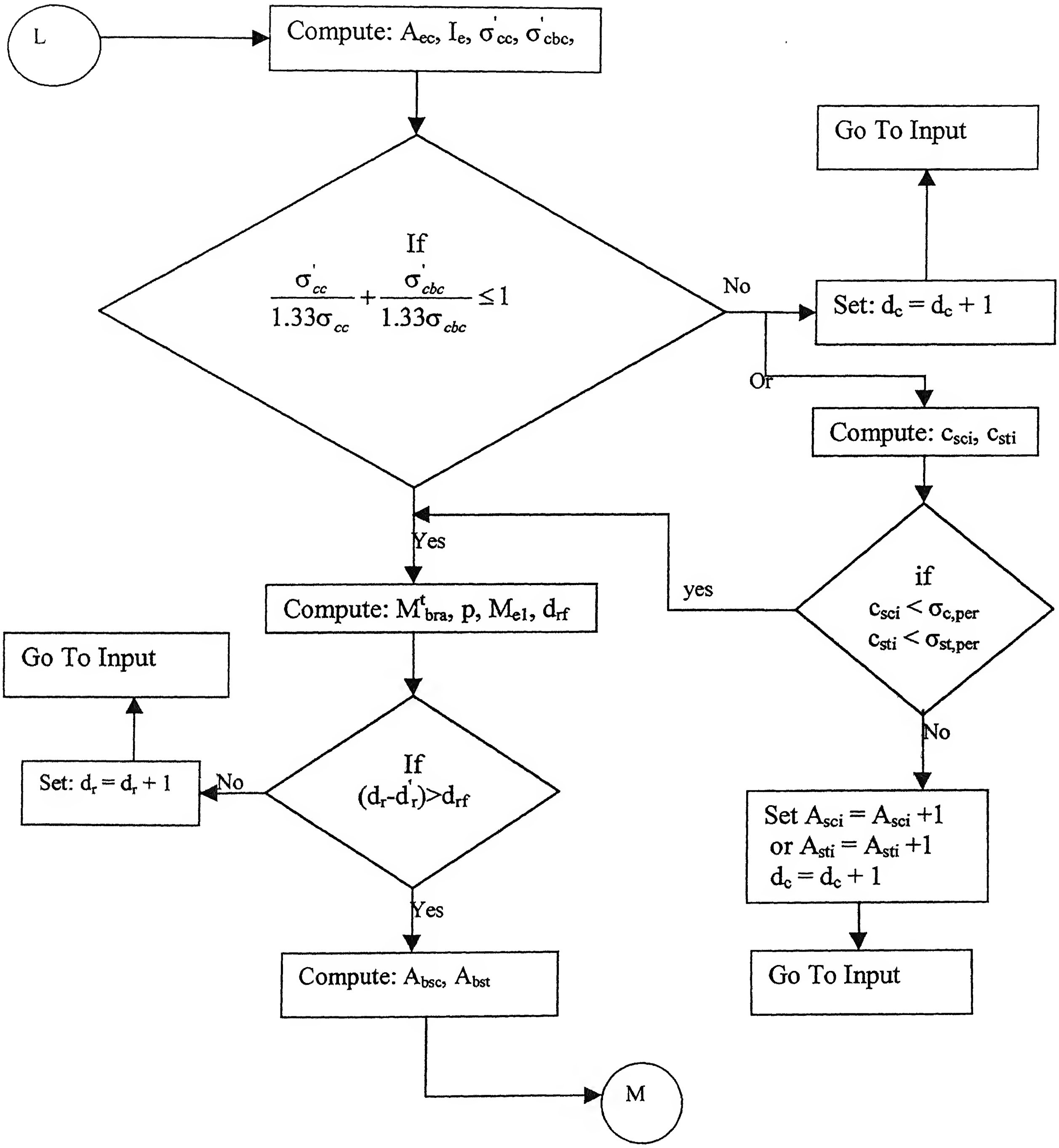


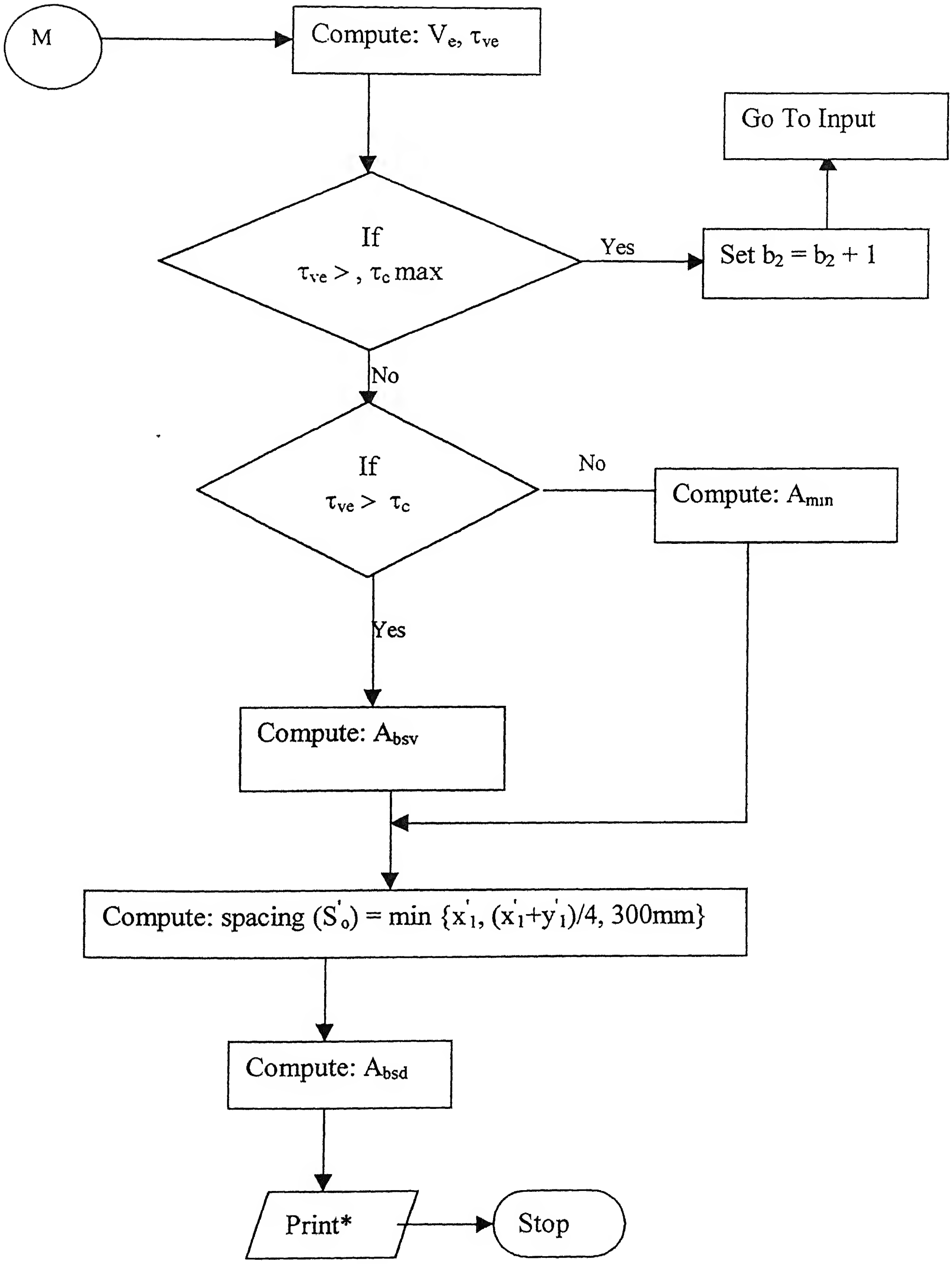

I
Compute: p_y, A_1, A_2, A_3, A_4, W_{wd}, W_{ww}, W_{wcd}, W_{wlb}, h_3, W_{pan}., W_{tpan},
Set: i = 1
If $i < n_1$
Yes
Compute: $Q^w{}_i$, $M^w{}_i$, V_i, q_i, $q_{i, max}$, $S_{i, max}$, $M_{i, max}$
Set: i = i+1
No
Compute: $Q^w{}_i$, $M^w{}_i$, V_i, q_i, $q_{i, max}$, $S_{i, max}$, $M_{i, max}$
J



J
Compute: θ, m_{n1}, $S_{b,max}$, M^t_b
Set i = 1
$1/K_{axial} = 0$
If
$i \le n_1$
Yes
Set:
$\frac{1}{K_{axial}} = \frac{1}{K_{axial}} + \frac{2}{nA_cE_cr_c^2}H^2x^2$
No
If
i = 1
Yes
$K_1 = K_{u,f,panel}$
No
Compute:
$K_{i,f,panel} = K_{i,f,panel} + \frac{12E_cI_cn}{x^3}\left[\frac{\frac{E_bI_b}{L}}{\frac{E_bI_b}{L} + \frac{2E_cI_c}{x}}\right]$
i = 2….n_1-1
No
If i = n_1
Set i = i + 1
Yes
$K_{n1} = K_{b,f,panel}$
K


K
Compute:
$\frac{1}{K_{u,b}} = \frac{1}{K_1} + \frac{1}{K_{n1}}$
Compute:
$\frac{1}{K_{flexural}} = \frac{1}{K_{u,b}} + \frac{1}{K_i}$
Compute:
$\frac{1}{K_s} = \frac{1}{K_{flexural}} + \frac{1}{K_{axial}}$
Δ, T, α_h, F_{lat}
If
$i < n_1$
Yes
Compute: F_i, V_{bi}, M_{bi}, V_c, M_c
Compute: F_i, V_{bi}, M_{bi}, V_{ce}, M_{ce}
Compute: $M_{col} = \max(M_{ce}, M_{I,max})$, $M_{bra} = \max(m_{n1}, M_{bi,max})$,
$V_{bra} = \max(S_{b,max}, V_{bi,max})$,
L


L
Compute: A_{ec}, I_e, σ'_{cc}, σ'_{cbc},
If
$\frac{\sigma'_{cc}}{1.33\sigma_{cc}} + \frac{\sigma'_{cbc}}{1.33\sigma_{cbc}} \leq 1$
No
Set: $d_c = d_c + 1$
Go To Input
Or
Compute: c_{sci}, c_{sti}
if
$c_{sci} < \sigma_{c,per}$
$c_{sti} < \sigma_{st,per}$
yes
No
Set $A_{sci} = A_{sci} + 1$
or $A_{sti} = A_{sti} + 1$
$d_c = d_c + 1$
Go To Input
Yes
Compute: M^t_{bra}, p, M_{e1}, d_{rf}
If
$(d_r - d'_r) > d_{rf}$
No
Set: $d_r = d_r + 1$
Go To Input
Yes
Compute: A_{bsc}, A_{bst}
M

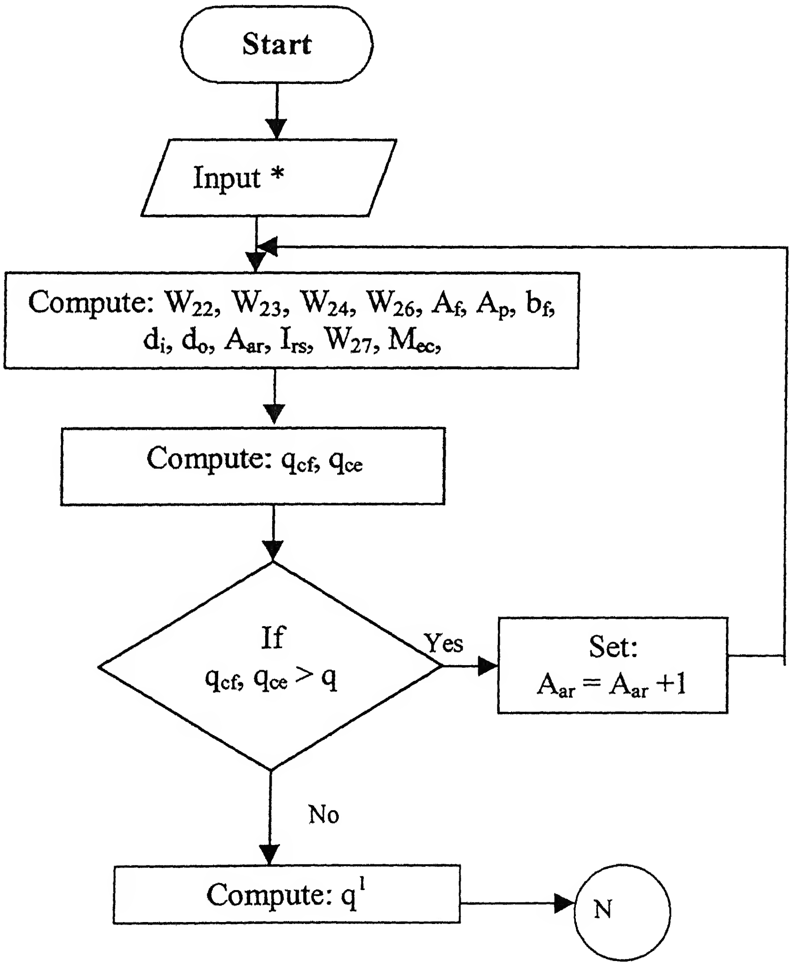

M
Compute: V_e, τ_{ve}
Go To Input
If
τ_{ve} > , τ_c max
Yes
Set $b_2 = b_2 + 1$
No
If
$\tau_{ve} > \tau_c$
No
Compute: A_{min}
Yes
Compute: A_{bsv}
Compute: spacing (S'_o) = min $\{x'_1, (x'_1+y'_1)/4, 300mm\}$
Compute: A_{bsd}
Print*
Stop

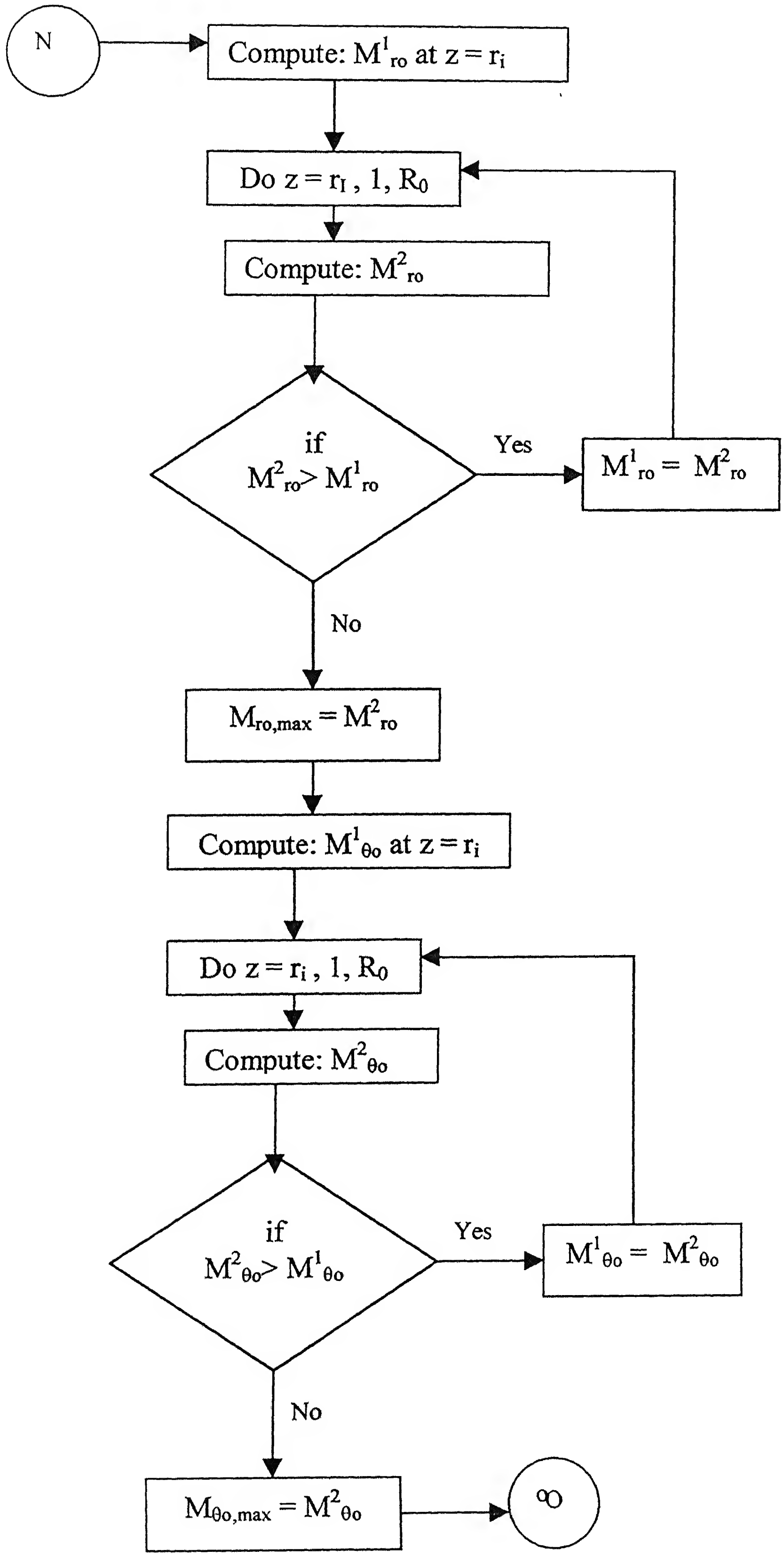
*__Input:__ Out put from section (4.2.1 & 4.2.2), y, n_1, x_1, d_c, γ_c, b_r, d_r, V_y, β_1, β, I, F_0, S_a/g,

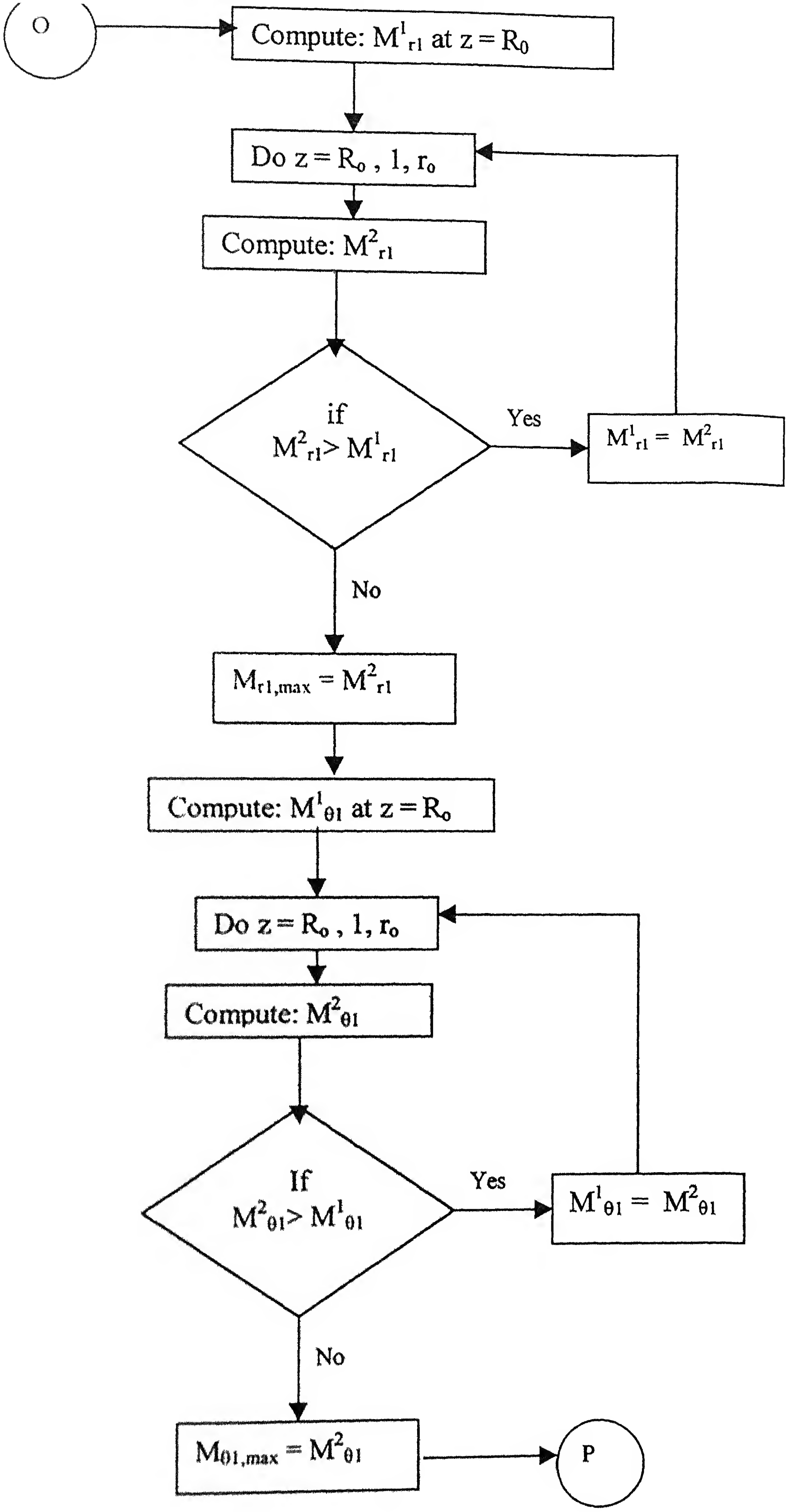
*__Print:__ All forses and stress in members, Area of reinforcement, Spacing of Reinforcement, Dimension of members.

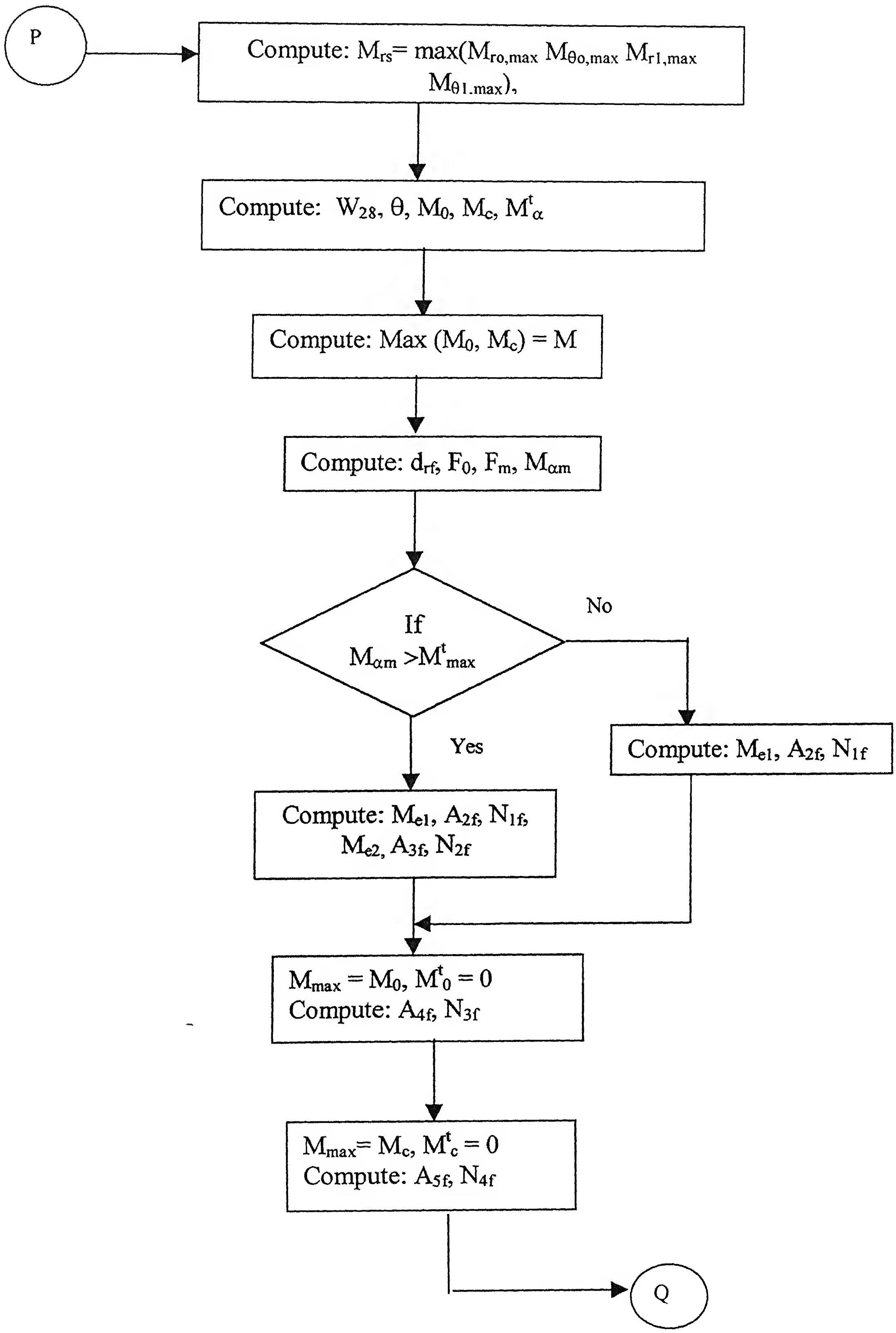
4.4 Algorithm for design of foundation

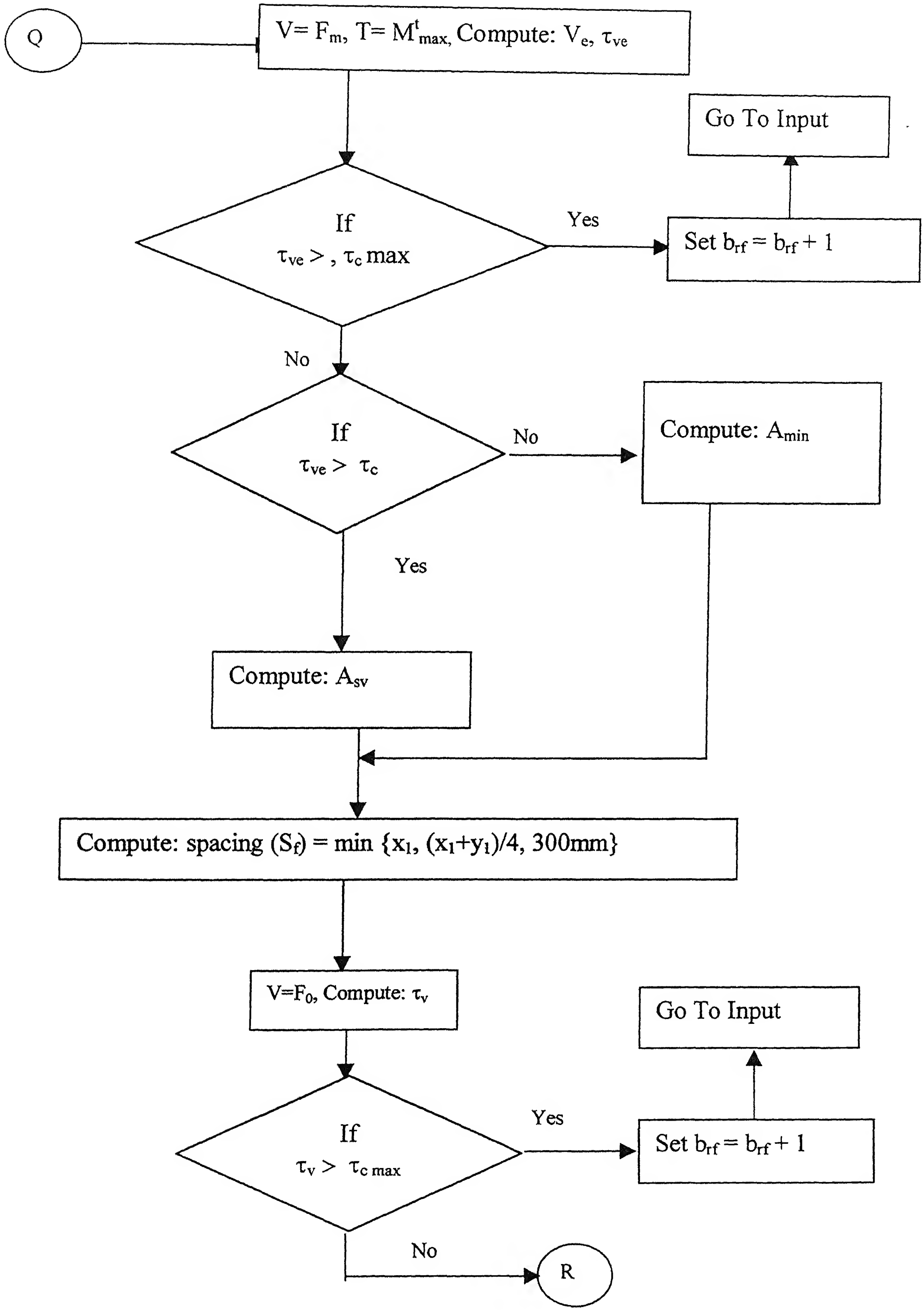
After designing the superstructure, the last step is designing the foundation of water tank. The maximum load coming from the superstructure is taken for the design of foundation. The following flowchart can be used to find the dimension and reinforcement of foundation, which is given in detail below.

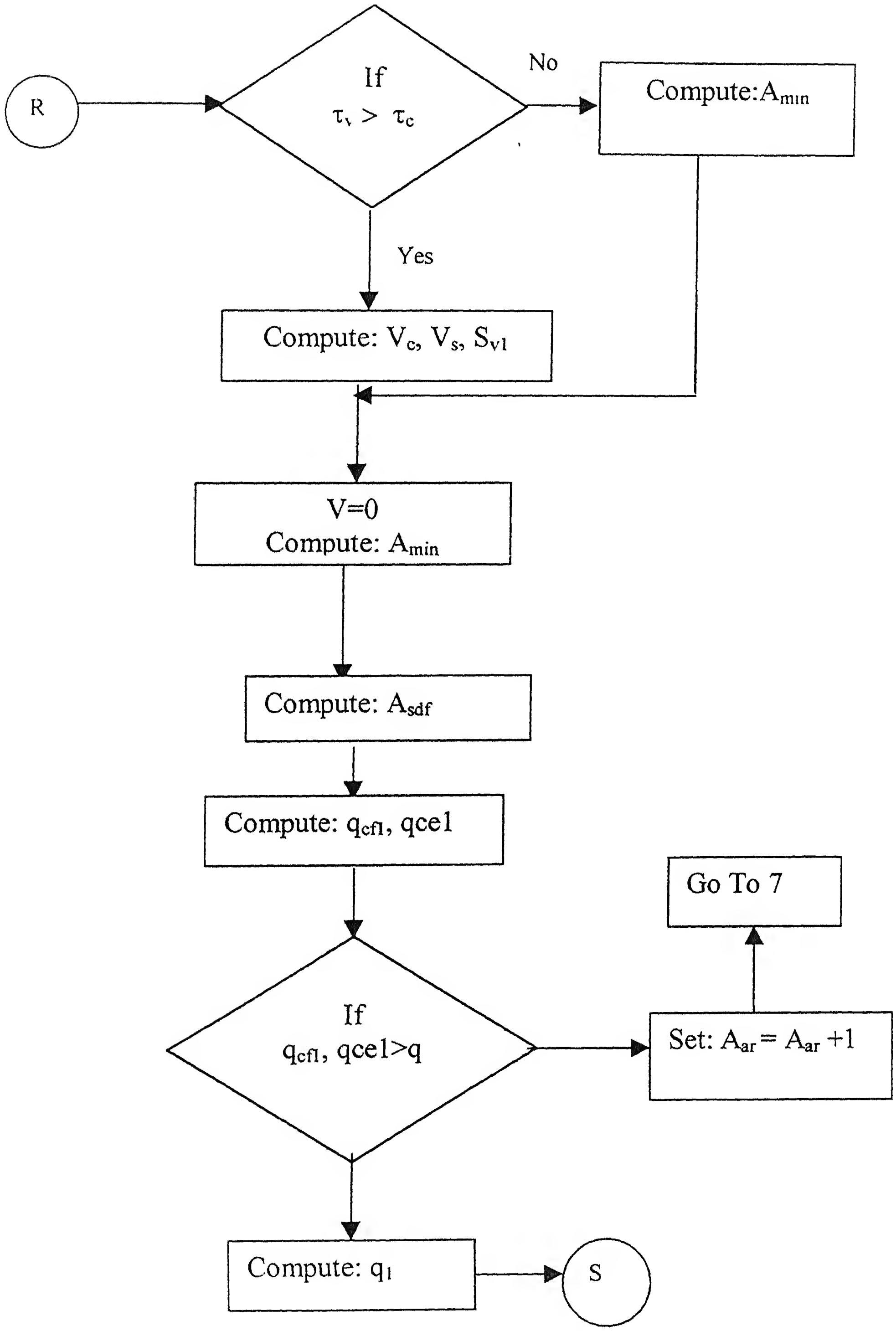


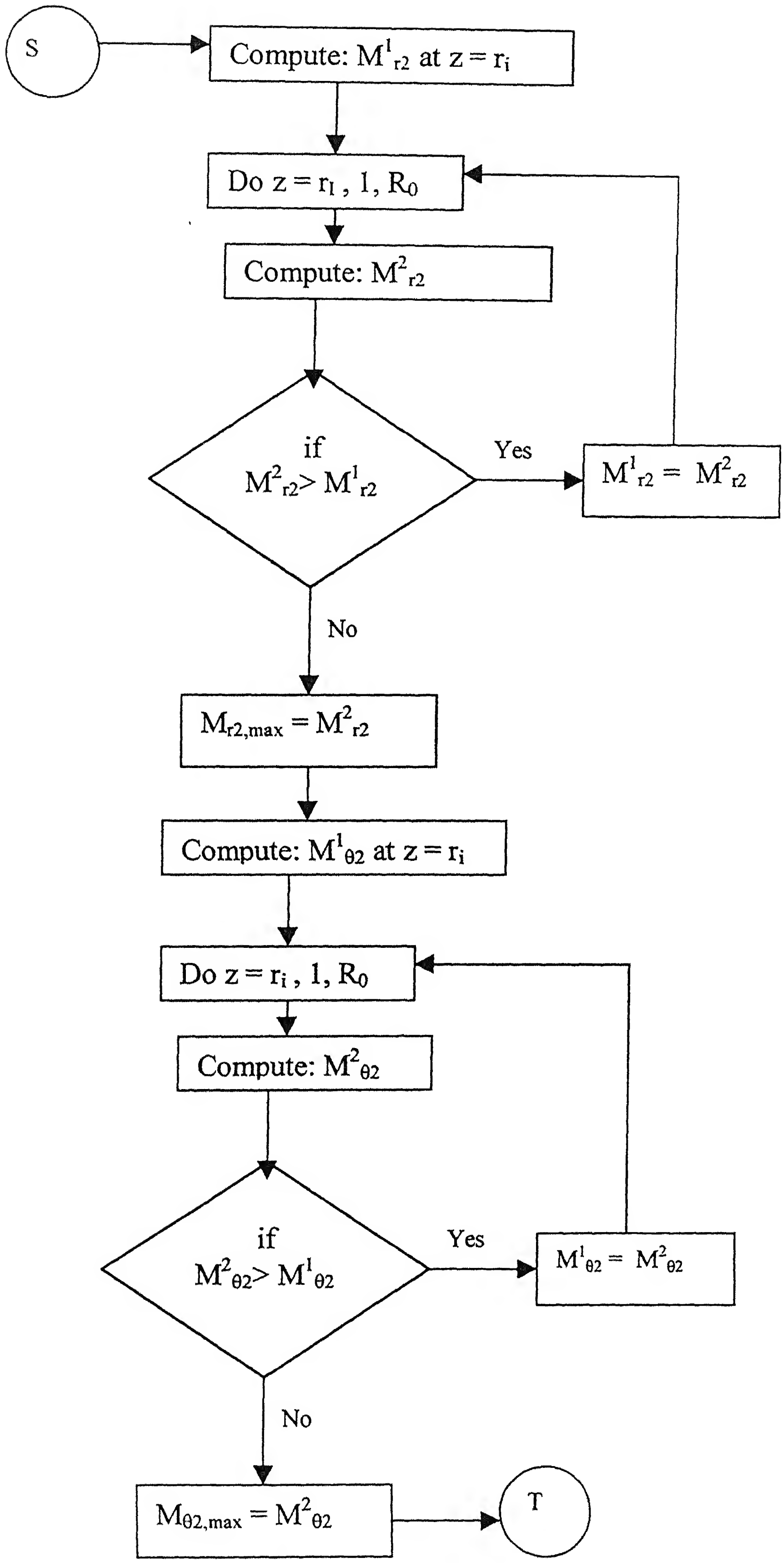

N
Compute: M^1_{ro} at $z = r_i$
Do $z = r_I$, 1, R_0
Compute: M^2_{ro}
if
$M^2_{ro} > M^1_{ro}$
Yes
$M^1_{ro} = M^2_{ro}$
No
$M_{ro,max} = M^2_{ro}$
Compute: $M^1_{\theta o}$ at $z = r_i$
Do $z = r_i$, 1, R_0
Compute: $M^2_{\theta o}$
if
$M^2_{\theta o} > M^1_{\theta o}$
Yes
$M^1_{\theta o} = M^2_{\theta o}$
No
$M_{\theta o,max} = M^2_{\theta o}$
∞

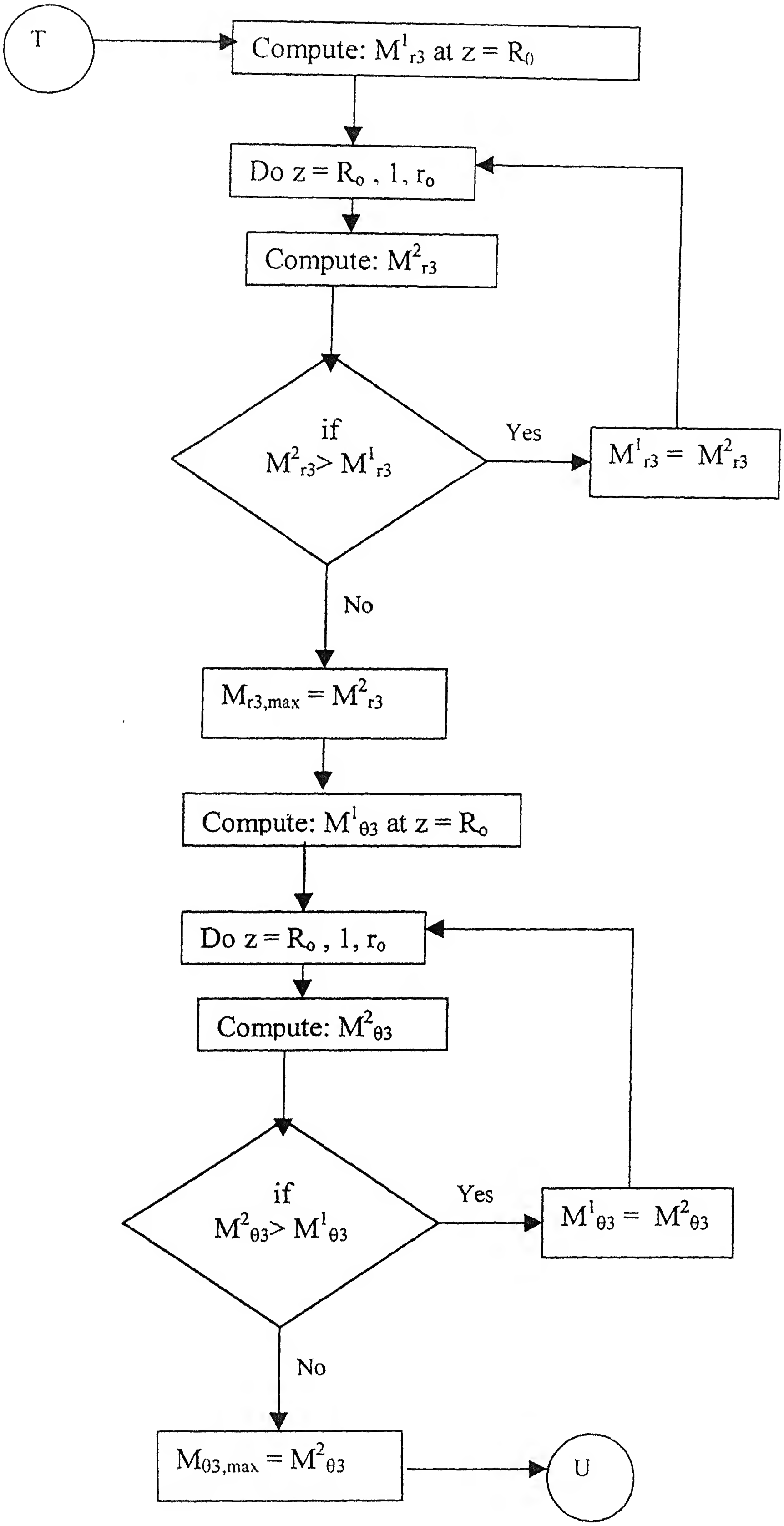

O
Compute: M^1_{r1} at $z = R_0$
Do $z = R_o$, 1, r_o
Compute: M^2_{r1}
if
$M^2_{r1} > M^1_{r1}$
Yes
$M^1_{r1} = M^2_{r1}$
No
$M_{r1,max} = M^2_{r1}$
Compute: $M^1_{\theta 1}$ at $z = R_o$
Do $z = R_o$, 1, r_o
Compute: $M^2_{\theta 1}$
If
$M^2_{\theta 1} > M^1_{\theta 1}$
Yes
$M^1_{\theta 1} = M^2_{\theta 1}$
No
$M_{\theta 1,max} = M^2_{\theta 1}$
P

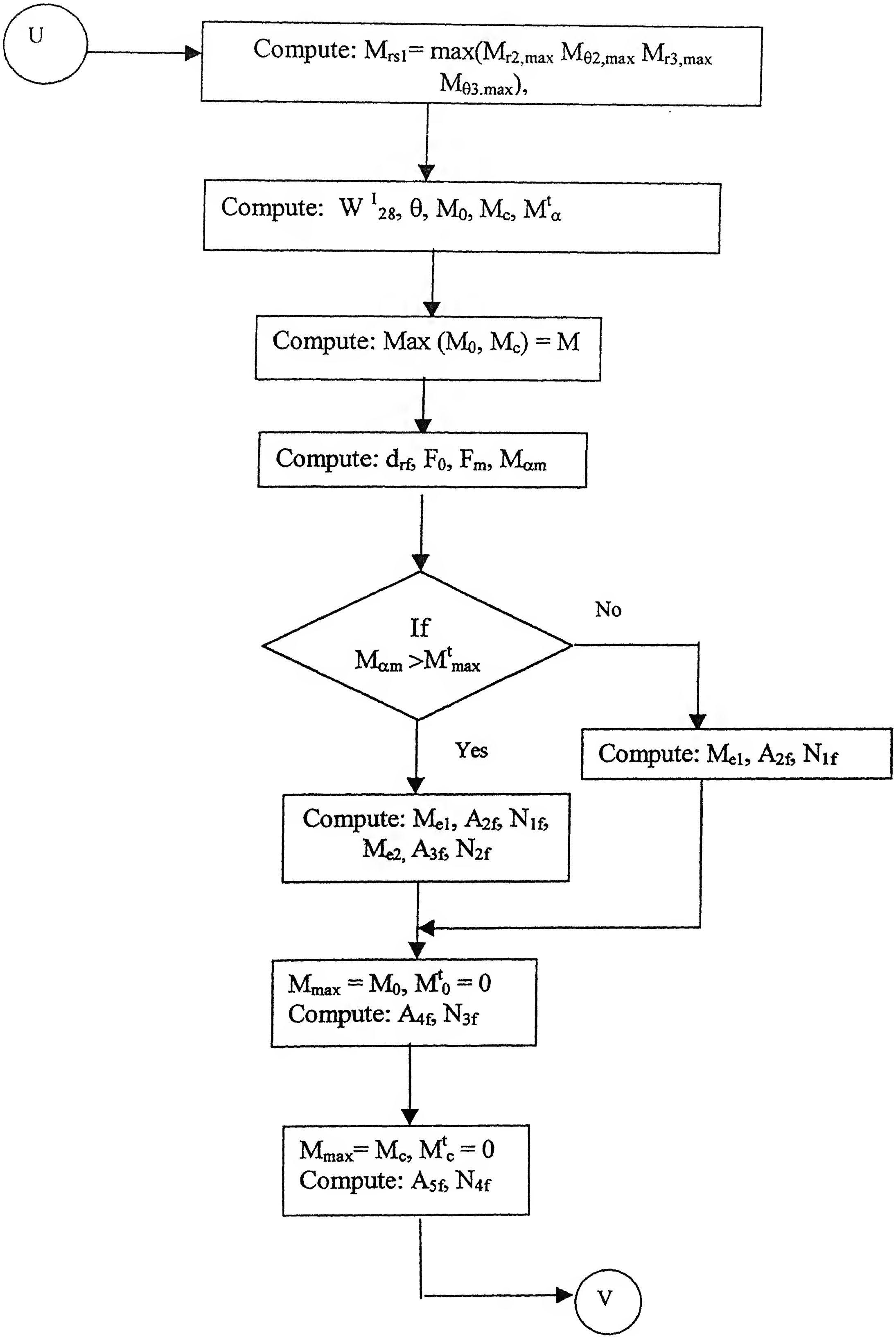

P
Compute: M_{rs}= max($M_{ro,max}$ $M_{\theta o,max}$ $M_{r1,max}$ $M_{\theta 1.max}$),
Compute: W_{28}, θ, M_0, M_c, M^t_α
Compute: Max (M_0, M_c) = M
Compute: d_{rf}, F_0, F_m, $M_{\alpha m}$
If $M_{\alpha m} > M^t_{max}$
No
Yes
Compute: M_{e1}, A_{2f}, N_{1f}
Compute: M_{e1}, A_{2f}, N_{1f}, M_{e2}, A_{3f}, N_{2f}
$M_{max} = M_0$, $M^t_0 = 0$
Compute: A_{4f}, N_{3f}
M_{max}= M_c, $M^t_c = 0$
Compute: A_{5f}, N_{4f}
Q

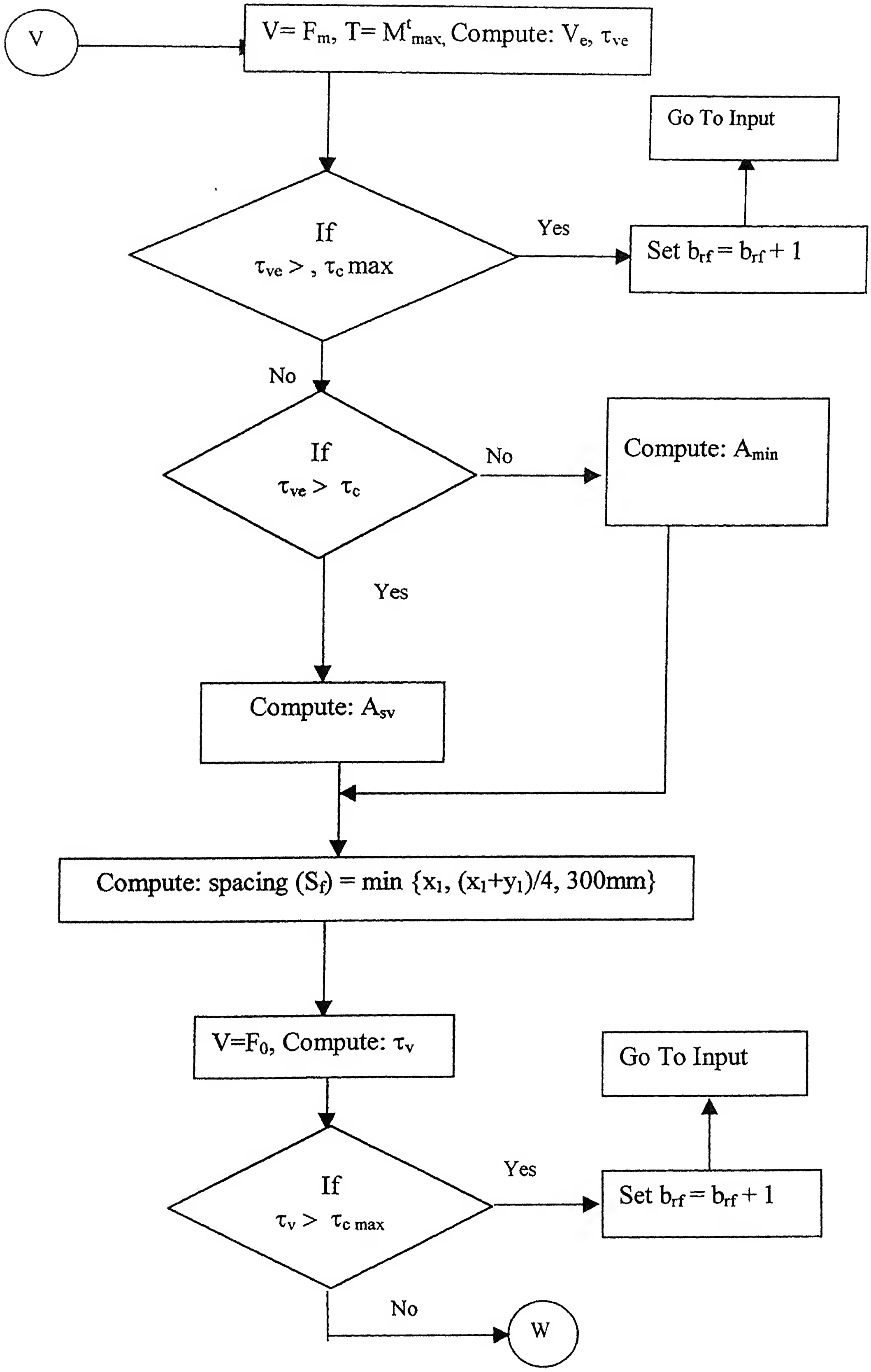

Q
V= F_m, T= M^t_{max}, Compute: V_e, τ_{ve}
Go To Input
If
τ_{ve} > , τ_c max
Yes
Set $b_{rf} = b_{rf} + 1$
No
If
$\tau_{ve} > \tau_c$
No
Compute: A_{min}
Yes
Compute: A_{sv}
Compute: spacing (S_f) = min {x_1, $(x_1+y_1)/4$, 300mm}
V=F_0, Compute: τ_v
Go To Input
If
$\tau_v > \tau_{c\,max}$
Yes
Set $b_{rf} = b_{rf} + 1$
No
R

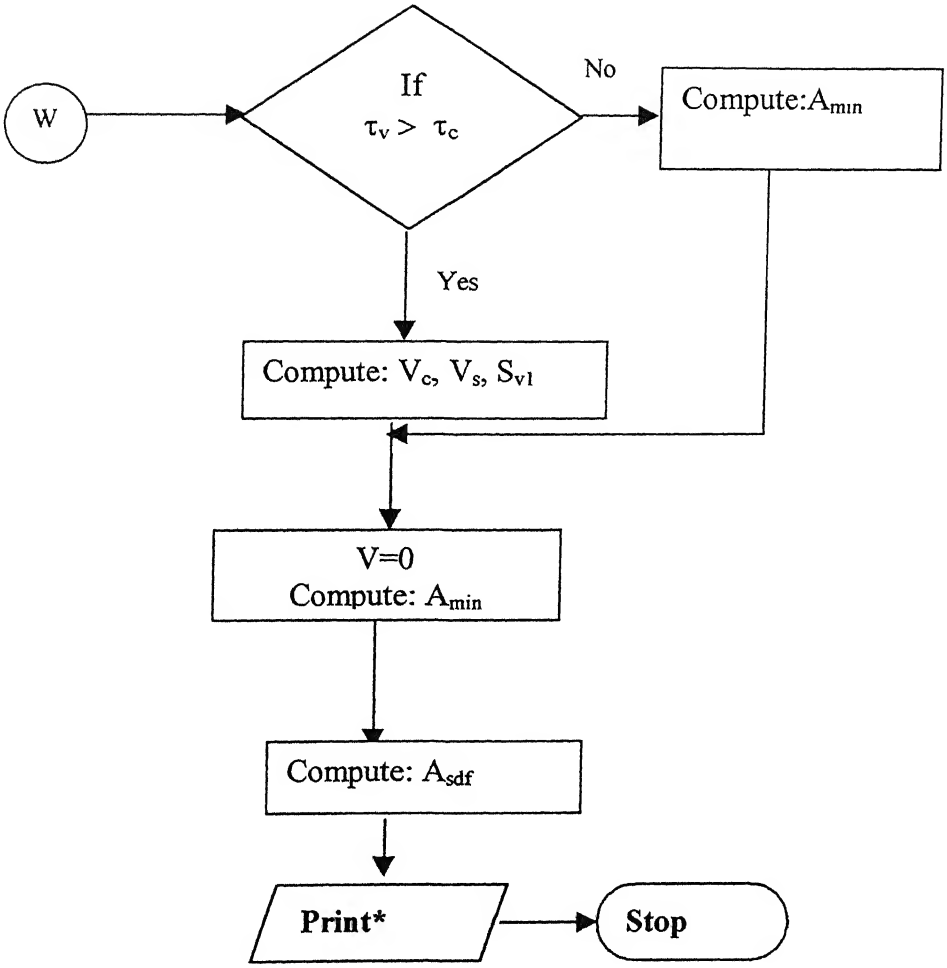

R
If
$\tau_v > \tau_c$
No
Compute:A_{min}
Yes
Compute: V_c, V_s, S_{v1}
V=0
Compute: A_{min}
Compute: A_{sdf}
Compute: q_{cfl}, qce1
Go To 7
If
q_{cfl}, qce1>q
Set: $A_{ar} = A_{ar} + 1$
Compute: q_l
S


S
Compute: M^1_{r2} at $z = r_i$
Do $z = r_I$, 1, R_0
Compute: M^2_{r2}
if
$M^2_{r2} > M^1_{r2}$
Yes
$M^1_{r2} = M^2_{r2}$
No
$M_{r2,max} = M^2_{r2}$
Compute: $M^1_{\theta 2}$ at $z = r_i$
Do $z = r_i$, 1, R_0
Compute: $M^2_{\theta 2}$
if
$M^2_{\theta 2} > M^1_{\theta 2}$
Yes
$M^1_{\theta 2} = M^2_{\theta 2}$
No
$M_{\theta 2,max} = M^2_{\theta 2}$
T


T
Compute: M^1_{r3} at $z = R_0$
Do $z = R_o$, 1, r_o
Compute: M^2_{r3}
if
$M^2_{r3} > M^1_{r3}$
Yes
$M^1_{r3} = M^2_{r3}$
No
$M_{r3,max} = M^2_{r3}$
Compute: $M^1_{\theta 3}$ at $z = R_o$
Do $z = R_o$, 1, r_o
Compute: $M^2_{\theta 3}$
if
$M^2_{\theta 3} > M^1_{\theta 3}$
Yes
$M^1_{\theta 3} = M^2_{\theta 3}$
No
$M_{\theta 3,max} = M^2_{\theta 3}$
U


U
Compute: $M_{rs1} = \max(M_{r2,max}\ M_{\theta 2,max}\ M_{r3,max}\ M_{\theta 3.max})$,
Compute: W^{I}_{28}, θ, M_0, M_c, M^t_α
Compute: Max $(M_0, M_c) = M$
Compute: d_{rf}, F_0, F_m, $M_{\alpha m}$
If $M_{\alpha m} > M^t_{max}$
No
Yes
Compute: M_{e1}, A_{2f}, N_{1f}
Compute: M_{e1}, A_{2f}, N_{1f}, M_{e2}, A_{3f}, N_{2f}
$M_{max} = M_0$, $M^t_0 = 0$
Compute: A_{4f}, N_{3f}
$M_{max} = M_c$, $M^t_c = 0$
Compute: A_{5f}, N_{4f}
V


V
V= Fm, T= Mtmax, Compute: Ve, τve
Go To Input
If
τve > , τc max
Yes
Set brf = brf + 1
No
If
τve > τc
No
Compute: Amin
Yes
Compute: Asv
Compute: spacing (Sf) = min {x1, (x1+y1)/4, 300mm}
V=F0, Compute: τv
Go To Input
If
τv > τc max
Yes
Set brf = brf + 1
No
W



***Input**: Output from section (4.2.1 & 4.3), W^1_{n1}, W_7, W_{11}, W_{25}, q, A_Φ, C_1, C_2, C_3, τ_c, τ_{cmax}, σ_{st}, b_{rf}, d_{rf}.

***Print:** Dimensions of foundation, upword forces on foundation, forces on different members of foundation, area of reinforcement, spacing of reinforcement.

Chapter 5

Summary and Conclusions

In the present study a standard design algorithm for the design of Intze tank is developed based on the review of available codal recommendations, analysis methodology and design data. The algorithm has got the capability to produce complete design for Intze tanks of different capacities.

The algorithm is developed and presented in the following three parts:

The first part gives the sequential and step-by-step methodology for the selection of container dimensions, and the computations for stresses and reinforcements, including the provisions for the needed iterations, if any, to satisfy the limitations of stresses.

The second part starts with the inputs obtained from first part and, gives the subsequent sequential steps for the designs of columns and braces, with required iterations, whenever needed, to satisfy the limitations of stresses.

Finally, the third part of algorithm presents the step-by-step procedure for the complete design and related checking for soil bearing pressure and material stresses, using the inputs from the second step.

SCOPE FOR FUTURE WORK

1. Development of a computer programme based on the algorithm presented in this thesis.
2. Revision of the presently algorithm for incorporating other types of supporting frame.
3. Revision of the presently algorithm for incorporating other types of foundations.

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[2] Arya, A. S., Data for Analysis and Design of Circular Shell Structures- Part 1 to 5, The Indian Concrete Journal, August 1969, Vol. 43, pp. 285-291 and p. 310; January 1970, Vol 44, pp.17-22 and p. 34; February 1970, Vol 44, pp 21-82; June 1970, Vol 44, pp. 251-261 and p. 269; December 1971, Vol 45, pp 514-524.

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